CHAPTER

6

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INTRODUCTION TO SUBSTRUCTURE

This chapter deals with some of the more common types of substructures used for bridges, as well as approach slabs, retaining walls and embankment protection.

EMBANKMENT AND REVETMENT

Embankment is most often needed to economically transition the bridge to the roadway. Settlement and stability analyses are important considerations when determining the fill heights and the bridge length. Geotechnical information such as consolidation studies and slope stability analysis should be obtained during preliminary design when fill heights are a consideration.

When erosion of the embankment is a concern due to stream forces, the exposed embankment should be protected with flexible revetment or rip-rap. The type of protection used depends upon hydraulic factors at each site. The Hydraulic Section will normally recommend the type of protection to be used when the Hydraulic Bridge report is submitted.

END BENTS, APPROACH SLABS AND RETAINING WALLS

The end bent and approach slab provides a transition from the rigid bridge structure to the flexible roadway embankment. Normally 12 m approach slabs are used for slab span and girder bridges except in weak soil areas. In these areas, 30 m pile supported approach slabs are often required to achieve a reasonable transition. Wingwalls are normally used on girder bridges to contain the embankment adjacent to the end bent.

Often retaining walls are required to contain the embankment in areas where right-of-way is a constraint. Either cast-in-place or mechanically stabilized earth walls (MSE) may be used. In most cases MSE walls are found to be the most economical unless site conditions dictate otherwise.

PILE BENTS AND COLUMN BENTS

The most commonly used bent type is the pile bent. The pile bent generally consists of a cast-in-place concrete cap used with precast-prestressed concrete piles. Pile bents are limited in height due to the slenderness and buckling capacity of the piles. Pile bents are very economical and can be used for stream crossings, highway crossings and railroad crossings when aesthetics are not a consideration.

The next most common bent type is the column bent. The column bent generally consists of a cast-in-place concrete cap, column and footing supported by piling. The columns may also be supported by drilled shaft footings or may be directly connected to the drilled shaft

Column Bents come in a variety of configurations. The most common is the two-column bent with round columns, normally used on two lane bridges.

Hammerhead column bents are often used on ramp structures. For bridges with column heights greater than 15 m, tapered columns are often used for greater economy.

Inverted T-caps are sometimes used for aesthetics, but primarily, where vertical clearance dictates their need. Inverted T-caps should be used only with approval of the Bridge Design Engineer.

RIVER PIERS AND COFFERDAMS

Where bridges cross major stream or river crossings and pile bents are not feasible, the use of a column bent constructed with a cofferdam is most commonly used. For major Mississippi River Bridge crossings, caissons are commonly used. As an alternate, large diameter drilled shafts should be investigated when deemed appropriate for the site.

All bridge crossings subject to navigational traffic should be investigated for the appropriate protection system due to vessel impact. This may include placing the piers out of the channel when it is feasible, designing the piers for vessel impact, or placing protection systems such as fenders and/or dolphin islands around the piers. The AASHTO "Guide Specification and Commentary for Vessel Collision Design for Highway Bridges" should be referred to for more information.

PERMANENT OR TEMPORARY SHEET PILE WALLS

Steel sheet pile walls are commonly used for both permanent and temporary applications. For sites where a permanent bulkhead is needed such as a navigational waterway, permanent sheet pile walls are often used. They are commonly designed either as a cantilever or tied back wall.

When bridges are built by phased construction (split-slab) on an offset alignment, or adjacent to railroad tracks, temporary steel sheet pile walls are often used to retain the existing or new embankment. In order to temporarily retain embankment, where deemed appropriate, temporary MSE walls may be used as an alternate to steel sheet pile walls.

DRIVEN PILES

INTRODUCTION

The most commonly used driven pile types are precast-prestressed concrete, cast-in-place concrete, steel "H", steel pipe and timber. This section is a guide to methods of pile foundation design and details.

It is conceded that the problem of foundation analysis is a highly complex one and that sometimes experience and intuition will be the better part of analysis. In this light, if any analysis and soil boring interpretation is followed blindly, serious errors in estimating foundation capacities can result.

The bridge designer normally proposes the type and length of pile foundation during preliminary plans. The Geotechnical and Construction sections should be consulted both during preliminary and final design to review and make comments on the proposed pile lengths, pile type and field-testing.

PILE DESIGN

Soil borings are required on all bridge projects for which piling is involved. In cases where an existing bridge will be widened or replaced on an existing alignment, the engineer should evaluate the following information first from the existing bridge records.

a) Existing bridge borings.

b) Existing test pile reports.

c) Existing pile driving records.

On projects for which all or some of the above information is available and contains sufficient information for the design of the foundation, there will be no need to order new borings.

Should the existing information not be sufficient or is not available, new borings must be ordered. New borings must be ordered through our Geotechnical Section. Information on how to order new borings and a boring request form can be found in chapter 1 or you may contact the Geotechnical Section. Any existing deep boring data should be attached to the boring request. The date when the complete geotechnical data (borings, consolidation and settlement analysis) are needed, should be included in the request. In the absence of this information, priority will be established by the preliminary plan date.

Piles shall be designed using service loads excluding live load impact. The maximum pile loads (design load) should always be shown on the construction drawings, normally with bent details or on the pile data sheet. Piles can be designed as friction piles, bearing piles, or a combination of both. The weight of the pile is normally neglected except in special cases involving large diameter piles and when cofferdams and tremie seals are used. The pile lengths for both on-system and off-system bridges are determined using the appropriate safety factors which are selected based on field-testing, type and amount of soils data and geotechnical analysis, type of project, static and dynamic load test and method of modeling pile installation. See Field Testing for more information on safety factors.

In order to set the plan pile lengths, a static analysis to determine ultimate pile capacity is normally performed on each boring for friction and bearing capacity. Due to the variety of soil conditions, this manual will not attempt to describe the analysis procedures. The engineer should confer with the Geotechnical Section when performing a static analysis. The design engineer will normally use the total soil shear strengths determined from either unconfined compression tests or standard penetration tests to determine the pile friction and end bearing for various piles and loads. If economically feasible, the engineer should attempt to tip end bearing piles in very dense sands ($n > 50$ blows). The end bearing piles should penetrate a minimum of 1.5 m into the 50 blows count material.

The plan pile lengths are established when the design event ultimate pile capacity divided by the required factor of safety is equal to or greater than the design load.

Pile sizes should be proportioned so that the following criteria are met.

a) As a general rule, the maximum slenderness ratio of $^{L}/_{d} \leq 20$ should be maintained.

   L = pile unsupported length (mm). The unsupported length is measured down below the channel bottom or ground line accounting for estimated scour, if appropriate (1.5 m minimum), plus a distance to the assumed point of pile fixity. In general, pile fixity can be assumed at 1.5 m below scour line or ground line. See figure on page 25.

   d = the least dimension or diameter of the pile section (mm).

   The maximum unsupported pile lengths based on a $^{L}/_{d} = 20$ are as follows:

Pile Size(mm)	[1]Max.unsupported length (mm)
350	7000
400	8000
450	9000
600	12000
750	15000

b) If $^{L}/_{d} > 12$, batter exterior piles.

c) All pile friction above the estimated scour line should be ignored. A minimum of 1.5 m of scour shall be assumed.

d) Piles should be proportioned so that the maximum pile design load is less than the allowable axial compressive loads and the allowable lateral loads. The allowable axial compressive pile loads are shown on page number 6 (6).

When favorable soil conditions exist, the design pile load should approach the upper limit of the allowable load range. In cases where the soil strength is questionable, lower design loads should be used.

e) The design of laterally loaded piles is usually governed by the lateral movement criteria. Proprietary computer programs are available for lateral pile geotechnical analysis. In addition the pile must be able to resist the additional lateral loads structurally. In the absence of lateral pile test information or a more detailed lateral analysis as described above, the following allowable lateral loads may be used.

Pile Type	Allowable Lateral Pile Load (kN)
Timber	13.0 kN
Concrete	18.0 kN
Steel	22.0 kN

f) Pile splices for concrete piles may be required when pile order lengths are expected to exceed the maximum casting length shown on the Standard Detail CS-216. The plans will have an S-item for pile splices (per each) with the anticipated number that may be required. The item will be bugged noting that this item may be deleted if final order lengths are less than maximum casting lengths shown in the plans.

[1] Maximum unsupported lengths may be exceeded in special situations with the BDE's approval. However, l/d > 25 will require investigations for elastic stability as columns.

Table, Allowable Axial Pile Load

PILE TYPE	PILE SIZE	ALLOWABLE AXIAL COMPRESSIVE LOAD	STANDARD DETAIL NAME
[1]Precast Prestressed Concrete Piles (square)	350 400 450 600 750	310-490 kN 400-575 kN 454-665 kN 710-1065 kN 1155-1730 kN	[2]CS-216(M)
Precast Prestressed Cylinder Piles	1370	2000-2500 kN	1370 mm Prestressed Cylinder Pile
Cast-in-place Concrete Steel Pipe Piles	350 400	490 kN 755 kN	Concrete Pile Alternates
Cast-in-place Concrete : Tapered: Raymond Helcor or Corwell Monotube	 350, 400 350, 400 350, 400	 490 kN, 755 kN 490 kN, 755 kN 490 kN, 755 kN	Concrete Pile Alternates
Timber	Butt Dia. (500-300) Tip Dia. (225-125)	265 kN 355 kN (Special Cases)	N/A
[3]Steel "H" (common sizes)	HP250 × 62 HP250 × 85 HP360 × 108 HP360 × 132 HP360 × 152 HP360 × 174	495 – 650 670 – 885 855 –11:30 1050 –1385 1200 –1590 1375 –1820 –	N/A
Steel Pipe (other sizes available,check w/suppliers)	PP460 × 9.5 PP610 × 12.7 PP760 × 15.9	830 –1100 1475 –1952 2310 –3050	N/A

[1] The allowable loads are based on the formulae set up by the joint AASHTO and PCI for piles with f'c = 35MPa PSI. (F.S. = 4 $^{L}/_{D}$ = 25 and 10 respectively); The loads are further reduced to account for soil capacity (Additional F.S. = 1.6 to 2.2). For situations where $^{L}/_{D}$ is approaching zero, i.e,. footing piles, and very favorable soil conditions exist, the designer may consider loads 1.5 times higher than those shown.
[2] Refer to standard detail CS-216(M) for maximum pile casting lengths.
[3] The lower and upper values represent allowables of the pile area 0.25Fy and 0.33 Fy respectively. Loads above the lower value may be used only with approval of the BDE and must incorporate static and/or dynamic load tests to confirm satisfactory results.

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The following typical splices are approved to be used on precast concrete and steel piles.

Pile Type	Pile Size (mm)	Pile Splice Type
Precast Concrete	350, 750	Cement dowel
Precast Concrete,	350,400,450	Dyna-a-splice (proprietary)
Precast Concrete,	600	ABB (proprietary)
Steel	All sizes	Full penetration butt weld

Standard details are available for the precast concrete pile splices, except for ABB splices.

g) For piles with a diameter less than 0.6 m, an 8 m minimum pile penetration should be provided for stream crossings below the estimated scour elevation.

h) For pile diameters equal to or greater than 0.6 m, a 9 m minimum pile penetration should be provided for stream crossings below the estimated scour elevation.

Battering piles is an expensive process and should be specified only when necessary. The exterior pile in bents should be battered when the unsupported pile length is excessive. Battering footing piles provides the necessary lateral support that is sometimes required to resist excessive lateral loads transferred from the column to the footing. This is particularly true for short column bents due to cap shrinkage. Maximum batter is usually 1 on 4 for footing piles and 1 on 8 on pile bents.

The allowable strength of precast concrete piles is seldom, if ever, exceeded by the design loads. Pile handling and transportation govern the design, thus establishing the maximum casting length and pick up point locations.

Cast-in-place concrete piles are used primarily in south Louisiana and are designed either as friction piles or combination friction and bearing piles. When cast-in-place piles are included as an alternate for precast piles, the pile lengths should be set based on the cast in-place pile, and only precast piles will be allowed in the end bents.

When hard driving is anticipated, particularly where jetting is not desirable, such as footing piles, the designer should consider non-displacement piles such as open-ended pipe or H piles. The designer should discuss this with the Geotechnical section.

The pile group capacity will be considered in foundation analysis if the center to center spacing is less than three pile diameters. Under normal situations, this is not allowed.

The consolidation settlement shall be computed for all pile groups. The pile group settlement shall be the same as shown for drilled shafts on page 19.

All timber piles for permanent structures shall be treated timber according to the Standard Construction Specification. Coastal treatment should be considered for use at locations south of I-10/I-12 line. Where coastal treatment is used, it should be clearly specified in the plans. Temporary structures such as detour bridges shall use treated timber piles.

PILE DETAILS

1. The following pile standard details are available and shall be included in the plans when applicable.

Standard Detail	**Description**
CS - 216(M)	Precast-Prestressed Piles
Concrete Pile Alternates	Cast-in-place Concrete Piles
Pile Splice Details	Cement Dowel, Dyna-a-Splice

2. Steel pipe piles shall generally be driven with open ends only.

3. Pile lengths should normally be shown on the general bridge plan. Pile tips, cutoff elevations, pile lengths and design loads should be shown on a pile data table. In addition, a column for end of driving pile capacity should be provided.

4. For moderate to complex projects involving skews, horizontal curves, interchanges, etc., or where interaction with existing foundations is present, the plans should include a foundation layout. The layout must show bents and/or footing pile locations and referenced to the centerline or P.G.L. Existing structures or substructures that may conflict with the pile driving must be clearly shown. Boring locations as well as test piles, CPT probings, PDA monitor piles and indicator piles must also be shown.

FIELD TESTING PROGRAM

INTRODUCTION

For most bridge projects, some type of pile testing or pile monitoring is normally performed. Soil conditions, design loads, magnitude of project, pile types, and economics are some of the factors determining the type and extent of the field-testing program to be implemented. Prior to beginning the field-testing program, the contractor must submit the necessary pile driving equipment information and the pile installation plan for evaluation and approval. The pavement and geotechnical design section will evaluate the proposed equipment based on the wave equation analysis (GRLWEAP™) for each pile type and size required in the plans. The criteria used in the evaluation consist of the pile driving stresses and the number of hammer blows per 0.25 m at the required ultimate pile capacity. The equipment and installation method shall be such that the piles will obtain the required penetration without damage.

The field-testing program is initiated in order to confirm or revise the estimated plan pile lengths as well as the contractor's proposed pile driving operations and installation plan. Upon evaluation of the data and results of the tests/monitoring, the project engineer, Construction, Geotechnical, and Bridge Design sections concur on pile order lengths and installation plan which allow the contractor to begin the pile fabrication process.

Field verification of pile capacity may be eliminated when either of the following two (2) conditions exist:

1. The soil boring data is consistent and indicates a very dense bearing strata ($N > 50$) which should be a minimum of 5 m thick.

2. Static load calculations indicate with the appropriate factor of safety (normally 2.75), it will be more economical to extend the piling than to enter into a test pile program with shorter length piles.

In these cases, the plans will specify order length piles.

DEFINITION OF TERMS AND PLAN REQUIREMENTS

Test Piles

Test piles are driven in advance of the permanent piles and are used to determine the length of the foundation piles by applying static loads at predetermined intervals. They may also be used to further evaluate the contractor's proposed method of installation and equipment. They are generally located in close proximity to a boring with consideration given to accessibility to the site for the driving and loading equipment and should model the most critical subsurface conditions for the area for which they will control. In some cases this may require the contractor to excavate the test pile location or utilize a casing to eliminate side friction in the upper portion. If cone penetrometer tests are utilized, they will generally govern the final location and tip elevation of the test piles.

Test piles should be cast long enough to be redriven, if necessary, to the plan tip elevation of the piles at the nearest bent and also sufficiently long to permit static and dynamic monitoring with the Pile Driving Analyzer.

Test piles are an expensive item for bridges, particularly on small projects and therefore, should not be used indiscriminately.

In addition to testing maximum axial compression, test piles may also be used to test the uplift (tensile) capacity of the pile primarily used for footings in cofferdams with tremie seals.

The plans or specifications should clearly designate if test piles are to serve as permanent piles.

When an item for a Test Pile is given in the plans, it is generally accompanied by items for Loading Test Pile, Reloading Test Pile, and Redriving Test Pile.

Due to the inherent properties and characteristics of soils and soil/pile interaction, the plans should always contain an item for *Loading Permanent Piles* for those unanticipated situations.

Cone Penetrometer Test, (CPT)

CPT probings may be used to determine pile order lengths. The CPT probings are also used to determine the final location and final pile tip elevation of the Test Piles and Indicator Pile. One of the following notes should be placed in the **General Notes of the bridge plans** depending on if the Department will perform the work, (note a), or if the Contractor will perform the work**,** (note b).

a) CPT PROBINGS: CONE PENETROMETER TEST (CPT) PROBINGS WILL BE REQUIRED AT THE LOCATIONS NOTED IN THE GENERAL PLANS OR FOUNDATION LAYOUT AND AT TEST PILE AND INDICATOR PILE LOCATIONS. CPT PROBINGS WILL BE PERFORMED BY THE DEPARTMENT IN ACCORDANCE WITH SECTION 804.04 (I) OF THE SPECIFICATIONS.

b) CPT PROBINGS: CONE PENETROMETER TEST (CPT) PROBINGS WILL BE REQUIRED AT THE LOCATIONS NOTED IN THE GENERAL PLANS OR FOUNDATION LAYOUT AND AT TEST PILE AND INDICATOR PILE LOCATIONS. CPT PROBINGS WILL BE PERFORMED BY THE CONTRACTOR.

When the CPT probings are performed by the contractor, (note b), the maximum CPT penetration elevation should be specified in the plans as follows:

a) THE MAXIMUM CPT PROBING PENETRATION ELEVATION FOR ALL CPT PROBINGS SHALL BE___ *(elevation in meters).*

b) THE MAXIMUM CPT PROBING PENETRATION ELEVATION SHALL BE ___ *(elevation in meters).* FOR CPT PROBINGS TAKEN AT BENTS (##) THROUGH (##).

c) THE MAXIMUM CPT PROBING PENETRATION ELEVATION FOR TEST PILE (##) SHALL BE ___ *(elevation in meters).*

Pile Driving Analyzer, (PDA)

PDA may be used to monitor the pile driving installation of Test Piles, Indicator Piles, and Monitor Piles with the Pile Driving Analyzer (PDA). A note similar to the following note should be placed in the **General Notes of the bridge plans**. It should be modified as needed.

PDA MONITORING: PILE DRIVING ANALYZER (PDA) MONITORING WILL BE REQUIRED AT EACH TEST PILE, INDICATOR PILE, AND MONITOR PILE OR AS DIRECTED BY THE ENGINEER.

Indicator Piles

Indicator Piles may be used to determine the final pile order lengths. In this case, an Indicator Pile is driven in advance of the production piles. The difference between Indicator Piles and a standard test pile is that loading Indicator Piles is not anticipated. Indicator piles should be cast long enough to be redriven, if necessary, to the plan tip elevation of the piles at the nearest bent or as determined necessary by the CPT probing. Indicator Piles are usually piles tipped on marginal end bearing soils. The Indicator Pile is used to access pile driveability problems such as hard driving which may require jetting or predrilling and to assess the bearing capacity of marginal bearing soil. The Indicator Pile driving installation is monitored with the PDA to evaluate the pile driving equipment performance and to monitor the pile driving stresses. Pile bearing capacity and driving criteria will be developed from data obtained from the PDA monitoring. The location of these piles is generally based on the type and size of pile to be driven and the anticipated subsoil profile at each bridge structure. The indicator pile is paid for as a modified test pile (i.e. Item 804.07(A) Precast Concrete Test Pile (Indicator Pile)). Pay items for each type of indicator pile used should be shown in the plans.

Redriving of indicator piles is paid for under Item 804(14), Redriving Test Piles. If it is determined from the driving records and PDA monitoring that the indicator pile should be load tested, each load test shall be paid for under Item 804(12)(A).

The following note should be placed in the **General Notes of the bridge plans**.

> INDICATOR PILE: INDICATOR PILES WILL BE REQUIRED AT THE LOCATIONS SHOWN ON THE GENERAL PLANS OR FOUNDATION LAYOUT.

Monitor Piles

A Monitor Pile may be used to monitor the pile driving installation with the PDA. This is usually accomplished by monitoring the first permanent pile of its type and size to be driven at each bridge structure or at a specified bent location. The location of these Monitor Piles is generally based on the type and size of pile to be driven and the anticipated subsoil profile at each bridge structure. The PDA is used to evaluate the pile driving equipment and to monitor the pile driving stresses. Pile driving criteria will be developed from this information. The Monitor Pile is paid for as a permanent pile. The dynamic monitoring is paid for with the PDA Monitoring item. One of the following notes should be placed in the **General Notes of the bridge plans** depending on where the indicator pile(s) is located.

a) MONITOR PILES: THE FIRST *(size, type)* PILE DRIVEN AT EACH BRIDGE SHALL BE MONITORED WITH THE PILE DRIVING ANALYZER (PDA).

b) MONITOR PILES: THE FIRST (size, type*)* PILE DRIVEN AT BENT NO. *(##)* SHALL BE MONITORED WITH THE PILE DRIVING ANALYZER (PDA).

c) MONITOR PILES: THE FIRST (size, type) PILE DRIVEN AT BENTS *(##)* THROUGH *(##)* SHALL BE MONITORED WITH THE PILE DRIVING ANALYZER (PDA).

Permanent Piles

Permanent Piles are those piles that are furnished by the contractor in accordance with an approved order list for use in production driving of foundation piles for the final substructure. If the driving resistance of a permanent pile should not correlate with the test pile or be less than that of the test pile, the engineer may require a static load test among other courses of action.

The following plan note should be placed in the **General Notes of the bridge plans**.

> PILES: ALL PILE REQUIREMENTS INCLUDING SIZE, TYPE AND MAXIMUM DESIGN LOAD AND TEST PILE REQUIREMENTS AS TO LOCATION AND TEST LOADING SHALL BE AS DESCRIBED ON THE PLANS OR IN THE SPECIFICATIONS. SEE STANDARD DETAIL CS 216(M). THE MINIMUM PILE TIP ELEVATIONS WILL BE PLAN PILE TIP ELEVATIONS UNLESS NOTED ON THE PLANS OR OTHERWISE AS DIRECTED BY THE ENGINEER.

In addition, one of the following pile length notes shall be included as needed in the **PILES:** note in the plans.

a) PILE LENGTHS SHOWN IN THE PLANS ARE ORDER LENGTH PILES.

b) PILE ORDER LENGTHS WILL BE PROVIDED AFTER COMPLETION OF THE CPT PROBINGS AND/OR TEST PILE LOAD TESTING AND/OR INDICATOR PILE INSTALLATION AND EVALUATION AS REQUIRED BY THE PLANS.

Jetting

When appropriate, jetting may be used to facilitate pile installation. This practice shall be predominately used when hard driving is anticipated during pile installation of end bearing piles. Jetting should not be allowed for friction piles, piles in footing, header banks or where stability of embankment or other improvements may be endangered. When jetting is allowed or required, the following note shall be added to the Special Provisions

> JETTING: JETTING MAY BE REQUIRED IN ACCORDANCE WITH SECTION 804.05(I) AT (list of the locations).

The note above amends the Standard Specifications sub section 804.05(I), Water Jets.

For more specific information on this subject as well as appropriate plans and specifications requirements including pay items, refer to the latest DOTD's Standard Specifications.

Types of Field Load Testing

1. **Static Load Test:** This work consists of applying static loads at predetermined intervals to Test Piles and in some cases to Indicator Piles or Permanent Piles. The static loads are applied in increments of 10 to 15 percent of the design load and held for an interval of 5 minutes. The loads are increased until pile failure occurs or three times the design load is reached. The ultimate pile capacity is determined through the analysis of the load settlement curve then a safety factor of 2.0 is applied to determine adequacy of pile tip elevation. Test piles will be loaded unless otherwise directed by the engineer. Test piles shall remain undisturbed for at least 14 calendar days after driving, unless otherwise directed by the engineer, to required penetration before beginning loading operations.

2. **Dynamic Load Test:** This work consists of assisting the Department in obtaining dynamic measurements with the Department's Pile Driving Analyzer (PDA) of test piles, indicator piles, and permanent piles during initial pile driving and during pile restrikes. The cost of equipment mobilization or any delays due to dynamic monitoring shall be at no direct pay. To allow space for attachment of instrumentation when dynamic monitoring is specified on test piles, indicator piles, and monitor pile, the piles shall be long enough to allow access to the top 2.5 pile diameters or side dimension of the pile at the end-of-driving penetration. The dynamic monitoring shall be performed for the purpose of obtaining the ultimate pile capacity, pile driving stresses, pile integrity, and pile driving system efficiency.

Types Of Pile Capacities

1. **Static Load Test Capacity:** This is the computed ultimate pile resistance that we are anticipating during the static load testing of a test pile, indicator pile, or permanent pile. The soil resistance will depend on the as-driven conditions such as overburden of the scour zone, scour zone soil resistance if it has not been cased or excavated, etc. The reaction system shall be sized to resist three times this estimated Static Load Test Capacity shown in the plans or as directed by the engineer.

2. **Ultimate Pile Capacity**: This is the ultimate pile capacity that has been determined from either a static or dynamic test of a test pile, indicator pile, or permanent pile.

3. **Design Load:** Is the maximum computed working load that the pile foundation is anticipated to support during the life to the structure.

4. **Design Event Ultimate Pile Capacity:** This is the computed static ultimate pile resistance that should be available after neglecting the scour zone and accounting

for the pile installation method. Plan pile lengths shall be based on the Design Event Ultimate Pile Capacity along with the appropriate safety factor.

5. **End-Of-Driving Pile Capacity:** This is the computed static ultimate pile resistance that should be encountered during pile driving which must be overcome to reach the plan pile penetration depth where the design load can be obtained with an acceptable safety factor. The soil resistance to be overcome includes resistance from unsuitable layers and scour zone. The effects of temporary loss or increase in soil strength due to driving operations should be considered. Pile installation methods which alter the in place soil resistance such as jetting, preboring, etc. should also be taken into account when computing the end-of-driving pile capacity.

Safety Factors

Safety factors for design, SF_{Design}, and for construction control, SF_{EOD}, have been established as shown on the accompanying table. These safety factors are selected based on design and construction control factors. Design factors are quantity of subsurface information and geotechnical analysis, type of project, and type of construction control, etc. Construction control factors are the use of static or dynamic load tests and the use of either wave equation or dynamic formula to determine pile bearing capacity, etc. The Static Design Safety Factor, SF_{Design}, is used to compute pile order lengths based on the Design Event Ultimate Pile Capacity. The Construction Control Safety Factor, SF_{EOD}, is used to determine the required End-Of-Driving Pile Capacity for construction control.

SF_{Design} = Static Design Safety Factor

(Design Event Ultimate Pile Capacity)

SF_{EOD} = Construction Control Safety Factor

(End-Of-Driving Pile Capacity)

Pile Design and Construction Control Safety Factors

Construction Control Method		Static Design Safety Factor SF_{Design}	Construction Control Safety Factor SF_{EOD}
Field Testing	Test Pile with Static Load Test	2.00	2.00
	Indicator Pile with Dynamic Monitoring	2.50	2.50
No Field Testing	On-System: Permanent Pile with Wave Equation	2.75	2.75
	On-System: Permanent Pile with Gates Dynamic Formula	3.50	3.50
	Off-System: Permanent Pile with Wave Equation	2.50	2.50
	Off-System: Permanent Pile with Gates Dynamic Formula	3.00	3.00

DRILLED SHAFTS

INTRODUCTION

As an alternate to driven piling, for certain projects, drilled shafts can prove to be an economical foundation alternate. A drilled shaft consists of an augered hole that is filled with reinforced concrete to establish a foundation. Drilled shafts will only be used with the approval of the Bridge Design Engineer Administrator, or his designated representative.

In the past, drilled shafts have been primarily used in the northern part of the state, where hard clays are prevalent. Wilcox material is particularly suited for drilled shafts because of its strength characteristics, its cohesiveness and its impermeability. In some of these areas, drilled shafts should be used exclusively, however, in other areas, where their economy is questionable, they may be specified as an alternate to piling, requiring separate substructure details. These cases will be determined on a project by project basis. In the past few years, the Department has been gaining confidence constructing drilled shafts in the lower part of the State in sandy soils using the slurry displacement method.

It is always preferable to construct shafts in the dry, however this is not always possible. A test hole will be placed in most contracts so that the contractor can demonstrate his ability to construct a drilled shaft. If a dry hole cannot be maintained, the contractor will have to construct the drilled shafts using the slurry displacement method with approved slurry. Construction methods are dependent on soil conditions, and will conform to Section 814 of the Standard Specifications.

APPLICATION

1. Projects with obstructions that impede the pile driving operation. There is construction equipment available today to construct drill shafts under existing structures with vertical clearances as low as 4.0 m.

2. Projects requiring heavily loaded foundations.

3. Projects involving conflicts with utilities, existing drainage, roads, bridge structures and/or projects where vibrations and excessive noise cannot be tolerated.

4. Projects using large drilled shafts to take the place of footings and extend the shaft to the top of the substructure using conventional columns.

5. Projects which have hard clays, particularly when it is anticipated that precast concrete piles cannot be driven through the soil without a pilot hole that extends to within a few feet of the founding elevation.

DESIGN

1. All projects involving drilled shafts will be approved prior to design, by the Bridge Design Engineer Administrator.

2. As a general rule, size the drilled shaft for a compressive stress of **3 MPa** across the cross section of the shaft. However, the soil characteristics and the design requirements for the shaft to sustain both axial and lateral loads will determine the final size and length of the required shaft.

3. Side friction will be utilized, i.e., the shafts will be constructed with the casing extracted, unless otherwise approved by the Bridge Design Engineer Administrator.

4. For drilled shaft design, Class S concrete will be used and the concrete compressive strength will normally be limited to f'c = 18 MPa.

5. All drilled shafts, where the concrete or slurry is placed under water, will be constructed with access tubes to allow for cross-hole sonic logging, CSL. The test will determine if anomalies are present in the shaft, which may reduce its capacity. If the shaft has a reduced capacity the payment and/or rejection will be based on the CSL testing.

6. Drilled shafts used in abutments shall have a minimum diameter of 600 mm, however, a diameter of 750 mm is preferable.

7. A minimum reinforcement of 1% of the gross shaft area shall be extended to the bottom of the shaft.

8. Drilled shafts are available in 150 mm increments from 450 mm to 3600 mm. Some shafts may be available in the 4800 mm range.

9. Drilled shafts should be spaced center to center a minimum of three (3) times the shaft diameter, however, shafts can be placed closer if group capacity is accounted for in the foundation analysis.

10. Battered drilled shafts will not generally used.

11. Belled footings will not be used.

12. Detailed clearances for the reinforcement to the outside of the drilled shaft will be 75 mm for shafts with a diameter of 750 mm or less and 150 mm for shafts greater than 750 mm.

For further design information consult the Pavement and Geotechnical Design Section and AASHTO.

Design Criteria

Service load design will be used. The maximum shaft load shown in the plans should be broken into two components, dead load and other group loads. The effects of down drag or uplift should be accounted for in the loads.

The final shaft diameter and length will satisfy the following axial load, lateral load and consolidation settlement criteria:

Axial Load:

1. The shaft size and length will be designed based on the following:

   a) Without a test load; Three (3) times the maximum shaft service load, however if the soil conditions are consistent from boring to boring and the designer has a high degree of confidence in the predictability of capacity of the shaft, 2½ times the maximum load can be considered.

   b) With a test load; Two (2) times the maximum shaft service load

2. Short term settlement due to load transfer will be limited to the following:

   a) (Settlement resulting from two times the maximum shaft service load) ≈ 20 mm to 25 mm.

   b) (Settlement resulting from 1½ times the maximum shaft service load) ≤ 7 mm.

Lateral Load:

1. Design the shaft for lateral loads, beginning with the size and length of shaft determined from the axial load design. The potential for loss of lateral capacity due to scour will be considered in the design.

2. Lateral movement will be considered and based on the specific type of structure and the nature of the lateral loads.

Consolidation Settlement:

1. Consolidation information should be obtained at the time the deep foundation borings are taken. The Geotechnical Section should be advised that this project is a potential drilled shaft site and consolidation information is required.

2. Consolidation settlements will be limited to the following:

   a) Standard Structures - 50 mm maximum or 25 mm / 50 m of span length.

   b) Movable Structures - 15 mm total and 15 mm differential between adjacent substructure elements

<u>Construction</u>

1. Drilled shafts may be constructed using the dry, wet or casing method of construction, or a combination of methods. However, the preferred and most economical method is the dry method.

2. Drilled Shafts shall be constructed in accordance with the plans and Section 814 of the Standard Specifications for Roads and Bridges.

3. Types of Drilled Shaft Construction**:**

   a) Dry Method:
   The dry construction method shall be used only at sites where the groundwater level and soil conditions are suitable to permit construction of the shaft in a relatively dry excavation. And also where the sides and bottom of the shaft may be visually inspected by the Engineer prior to placing reinforcement and concrete. The dry method consists of drilling the shaft excavation, placing the reinforcing cage, and concreting the shaft.

   b) Wet Method:
   The wet construction method may be used at sites where a dry excavation can not be maintained for placement of the shaft concrete. This method consists of using slurry to maintain stability of the hole perimeter while advancing the excavation to final depth, placing the reinforcing cage, and concreting the shaft. Where drilled shafts are located in open water areas, exterior casings shall be extended from above the water elevation into the ground. The casing shall be installed in a manner that will produce a positive seal at the bottom of the casing so that no seepage of water or other materials occurs into or from the shaft excavation.

   c) Casing Method:
   The casing method may be used when shown on the plans or at sites where the dry or wet construction methods are inadequate to prevent caving or excessive deformation of the hole. In this method the casing may be either placed in a predrilled hole or advanced through the ground by twisting, driving or vibration before being cleaned out. In either case (b) or case (c), the designer determines that the casing must be permanent, then a separate pay item must be included in the plans.

4. Slurry Types:

a) Polymer Slurry**:**
This type of slurry is a polymer powder that is pre-mixed with potable water and placed in the drilled hole. The weight of the slurry is used to counteract the hydrostatic pressure from the surrounding soil formation. The polymer prevents caving of the sides of the hole and is destroyed during the placement of the concrete by the chemical reaction with the cement. The polymer slurry, once destroyed, normally leaves no residual material in between the concrete and the soil interface. The advantages of this type of slurry is that it may be easily disposed of.

b) Mineral Slurry**:**
This type of slurry puts soil particles in suspension and will form a membrane or a filter cake at the walls of the hole. The membrane acts to prevent caving or collapse of the hole provided the hydrostatic fluid pressures inside the hole exceed the pressures in the soil formation. This filter cake can reduce the perimeter load transfer of the shaft to the surrounding soil if left in place for extended periods of time. The advantage of the Mineral Slurry is that you can counteract a larger hydrostatic pressure. The disadvantage is that it must be recovered and disposed of as a hazardous waste.

PILE BENTS

INTRODUCTION

Pile bents consist of prestressed-precast concrete piles with a cast-in-place reinforced concrete cap. (Steel pipe piles or HP piles may be used in site-specific locations). The piles extend out of the ground to serve as columns and are imbedded into the bottom of the cap. The following is a guide for the structural analysis and the construction details for general use in the preparation of plans for pile bents. The AASHTO Specifications shall be adhered to except as amended or supplemented herein.

Commentary

Due to the high variable conditions of foundation soils and tolerances allowed or expected in the construction of pile bents, the analysis should be kept relatively simple to reflect this variability. Rigorous frame analysis on pile bents is also frustrated by the indeterminate amount of fixity where the pile enters the ground and at the juncture between the pile and the cap.

Double row pile bents may be considered when additional capacity or stability is desired. The particular uses of double row pile bents are to resist longitudinal live load movement of long trestle type structures, and to add stability for high level crossings or weak soil areas. This practice should be limited to the extent that if a small number of larger piles are required, a column bent with foundation piles may be more economical.

In the absence of a complete analysis, the designer should consider using one double row pile bent in every continuous unit. Where the $^L/_d$ ratio is approaching the maximum limit, set forth herein, the distance between double row pile bents shall not exceed 150 m. In cases where the $^L/_d$ ratio approaches the moderate range of 14 to 16, the distance between double row pile bents shall not exceed 200 m.

ANALYSIS

Traffic loads should be placed in accordance with the AASHTO Specifications, anywhere within the design traffic lanes to cause maximum or critical stresses in the bent structure. Live load impact shall be applied to pile bent caps in the analysis.

The general method used for the analysis of pile bents containing four (4) or more piles is simplified because of the indeterminate nature of the bent structure. The maximum cap moment is computed assuming a simple span between adjacent pile centers allowing for a maximum mislocation of piles. The positive cap moment due to dead load plus live load plus live load impact is adjusted by a 20% reduction for continuity and the resulting value is used for both positive and negative moments. The related simple span shears should be used unaltered.

If the designer prefers more precision, the cap may be considered as a continuous beam over the pile support points to get a more precise steel requirement. Bent caps with less than four (4) pile support points shall be considered as a continuous beam.

Unequal dead load reactions from adjacent spans will be placed on the bent cap such that there are no appreciable dead load moments caused in the longitudinal direction. This will be accomplished by shifting the joint. An effort should be made to space the piles equally and at the same time balance the dead load reaction such that there will not be more than 50% deviation between the interior and exterior pile dead loads.

The allowable structural limit of a pile is controlled by the slenderness ratio which in effect keeps the pile structurally "a short column" (where column buckling is not critical). It can be shown that the standard prestressed pile has tremendous reserve strength against axial failure. The implication is that the foundation soil, the practical span of the bent cap and the allowable slenderness ratio of the pile will influence the pile size more than the allowable structural limit of the pile. The measure of the pile slenderness ratio is $^L/_d$.

L = the unsupported length

d = the least dimension or diameter of the pile section

The unsupported pile length (L) is usually measured from the bottom of the cap to the ground line plus penetration into the ground. The point of fixity is dependent on judgement, accounting for the soil stiffness at the surface or predicted scour at stream crossing. The point, which the pile is considered fixed, shall not be less than 3 m below an existing streambed or 1.5 m below the ground surface outside the floodplain.

For slab spans, the live load shall be placed to cause a maximum reaction to the bent. The wheels on the bent may be treated as concentrated loads. The contributing reaction of the wheels on the slab span may be summed and equally distributed to all the piles in the bent as a uniform load to the bent cap. The same number and size of bars shall be used in the top and bottom of cap.

DESIGN DETAILS

1. Batter exterior piles when $^L/_d > 12$. L should account for the predicted scour depth and distance to pile fixity. A minimum of 1.5 m of scour should be accounted for as well as 1.5m to point of fixity.

2. Particular attention shall be given to battering the exterior piles on stream crossings and on bridges in a horizontal curve.

3. Pile batter shall be 1 on 8.

4. Maximum pile loads shall be determined from service loads applied without live load impact and should be shown on the bent details indicated on the bent details.

5. Double stirrups shall be used in all pile bent caps exceeding 750 mm in width.

6. Stirrups shall be spaced at a maximum of 300 mm. The stirrups adjacent to piles shall be located at a maximum of 75 mm from the face of the pile and the first space shall be a maximum of 150 mm. The size of stirrups shall be a minimum of No. 10 bars.

7. The centerline at the top of the exterior pile shall not exceed more than 460 mm beyond the centerline of the exterior girder.

8. The pile bent design should account for at least two (2) adjacent piles being mislocated 75 mm each or one(1) pile mislocated 150 mm in the direction parallel to the cap..

9. Pile bent caps shall have a minimum depth of 600 mm for all slab span supporting bents and all single row pile bents with less than 600 mm piles and 700 mm for all single row pile bents with 600 mm piles or larger. Double row pile bents shall have a minimum cap depth of 750 mm.

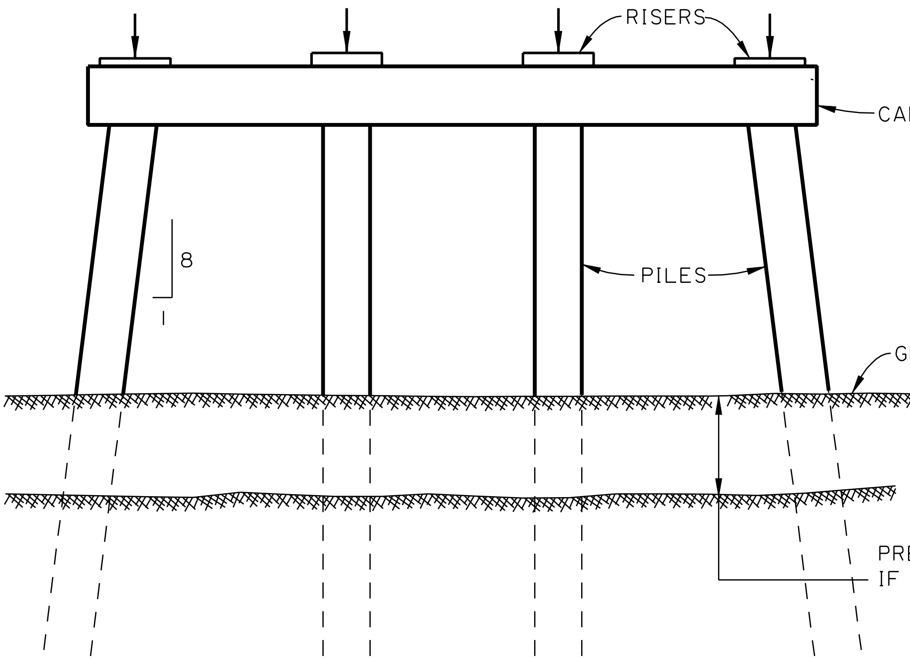
10. The minimum longitudinal cap steel shall be in accordance with the AASHTO Specifications.

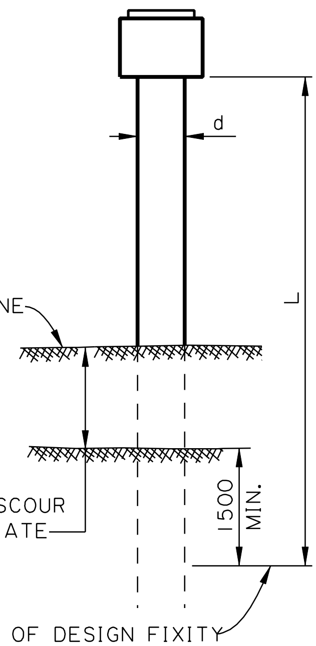
11. As a general rule, $^{L}/_{d}$ in pile bents should not be over 20. See further discussion under driven piles.

12. The top and bottom reinforcement in caps shall be the same.

13. The concrete quantity for the pile cap shall not include the volume of concrete displaced by the pile embedment.

14. Spacing for double row pile bents shall be determined on an individual basis.

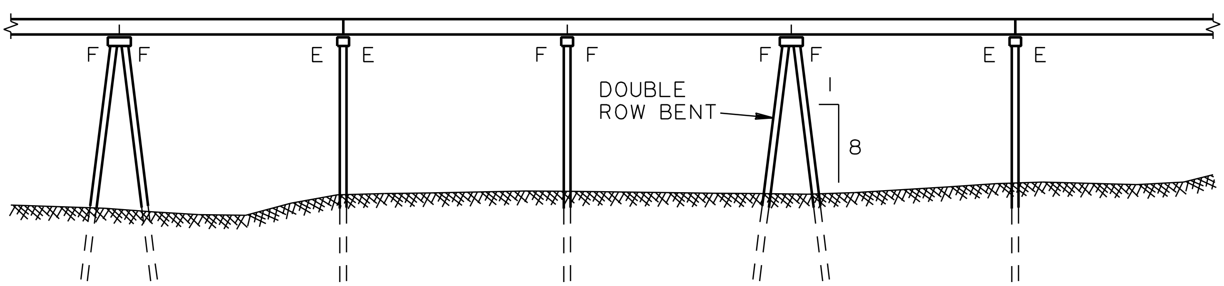




ELEVATION VIEW OF PILE BENT

(SINGLE ROW)

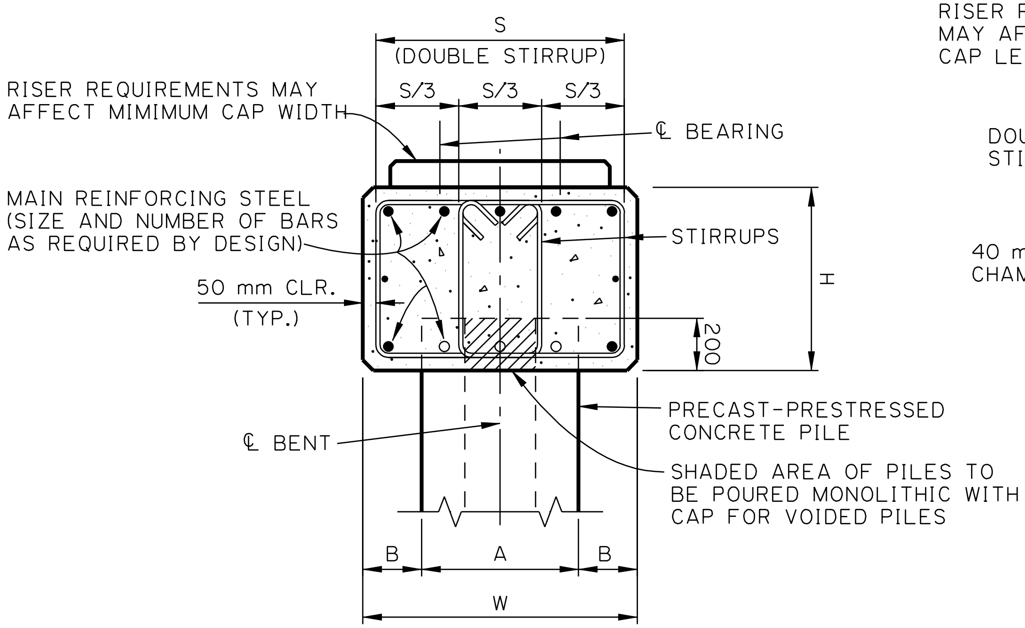
END VIEW



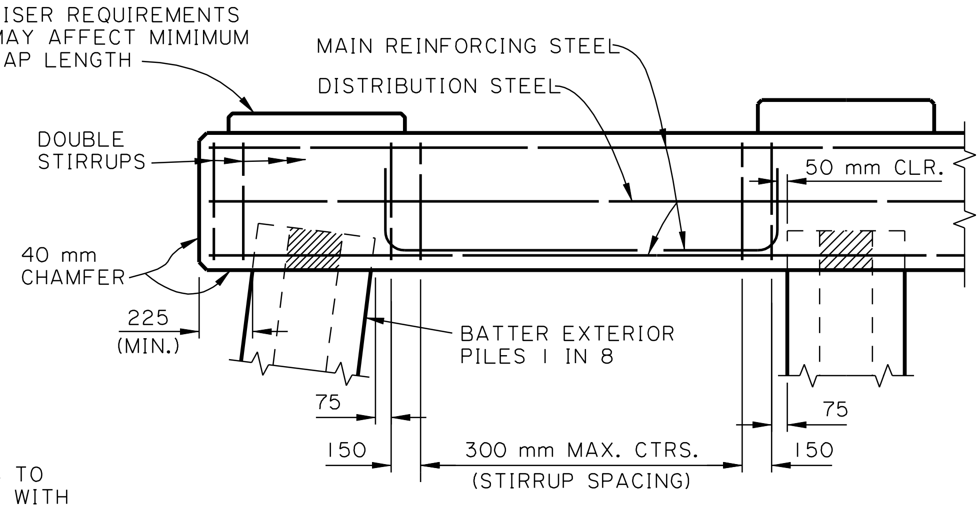
TYPICAL BRIDGE LAYOUT
WITH BATTERED PILE BENTS

PILE BENT TYPICAL LAYOUT

PILE BENT DETAILS



TYPICAL SECTION OF SINGLE ROW PILE BENTS



TYPICAL PILE BENT ELEVATION
(DOUBLE ROW OR SINGLE ROW)

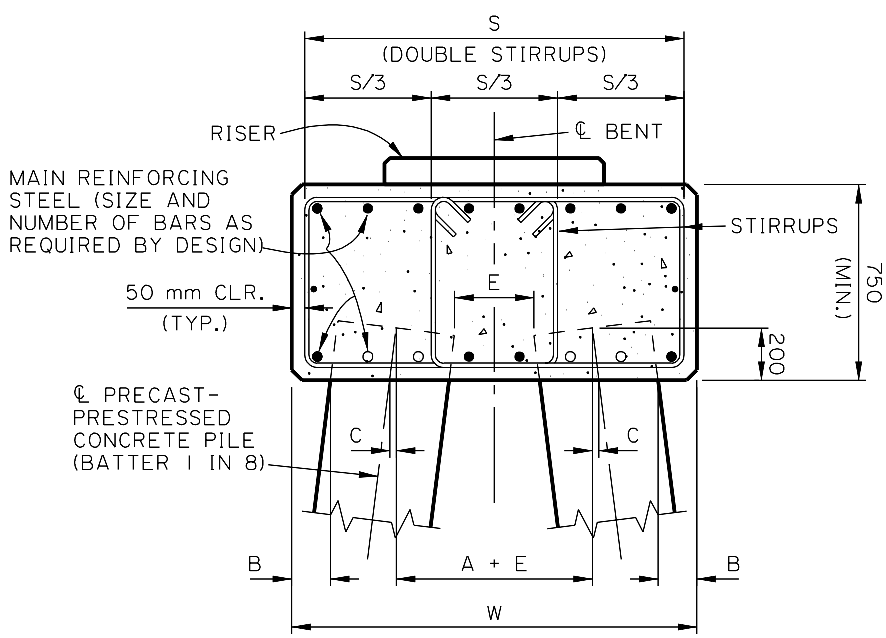
SINGLE ROW REQUIREMENTS			
A	B	W	H
350 Φ	150	650	600
400 Φ	150	700	600
450 Φ	150	750	600
500 Φ	225	950	600
600 Φ	225	1050	700
750 Φ	225	1200	700
900 Φ	225	1350	700

A = PILE SIZE
B = MINIMUM PILE TO CAP EDGE
W = MINIMUM CAP WIDTH = A+2B
H = MINIMUM CAP HEIGHT

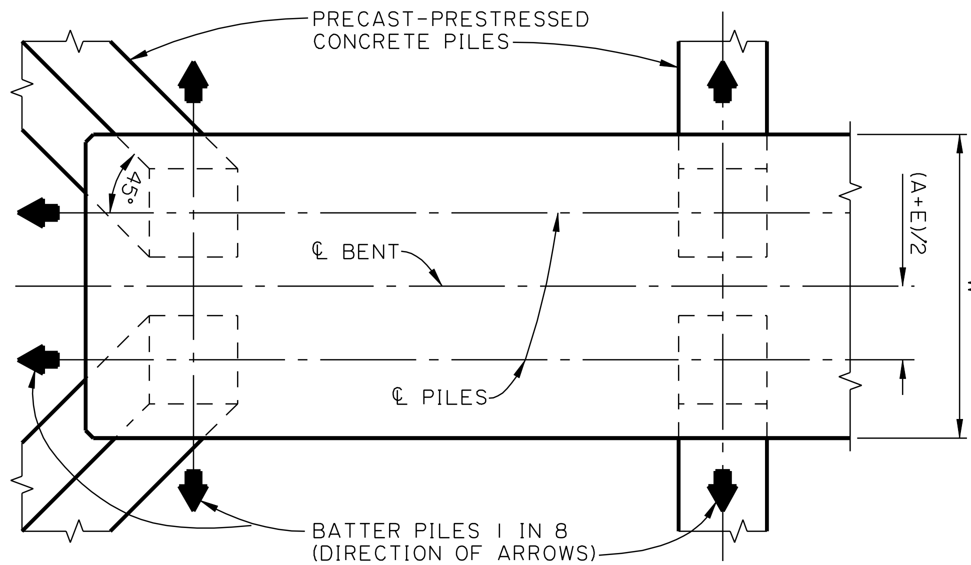
NOTE:

THIS IS A GENERAL CRITERIA AND WILL NOT SATISFY ALL CONDITIONS THAT MAY BE ENCOUNTERED. SPECIAL CONSIDERATION SHOULD BE GIVEN TO THE DESIGN OF THOSE PILE BENTS FOR WHICH THE CONDITIONS ARE UNUSUAL.

PILE BENT DETAILS



TYPICAL SECTION OF DOUBLE ROW PILE BENTS



PART PLAN SHOWING PILE BATTER

DOUBLE ROW REQUIREMENTS				
A	B	C	E	W
350 ɸ	150	25	300	1350
400 ɸ	150	25	300	1450
450 ɸ	150	25	300	1550
500 ɸ	225	25	350	1850
600 ɸ	225	25	350	2050
750 ɸ	225	25	350	2350
900 ɸ	225	25	350	2650

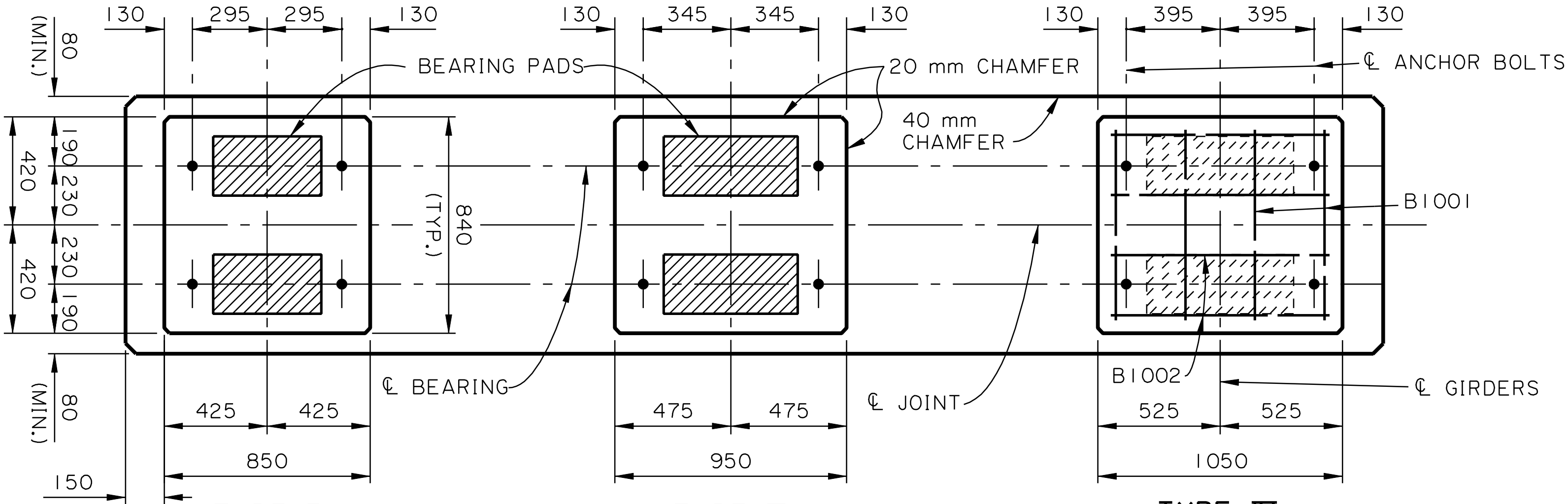
A = PILE SIZE
B = MINIMUM PILE TO CAP EDGE
C = BATTER OFFSET
E = MINIMUM CLEARANCE BETWEEN PILES
W = MINIMUM CAP WIDTH = 2A+2B+2C+E

NOTE:

THIS IS A GENERAL CRITERIA AND WILL NOT SATISFY ALL CONDITIONS THAT MAY BE ENCOUNTERED. SPECIAL CONSIDERATION SHOULD BE GIVEN TO THE DESIGN OF THOSE PILE BENTS FOR WHICH THE CONDITIONS ARE UNUSUAL.

DOUBLE ROW CONSIDERATIONS

PILE BENT DETAILS

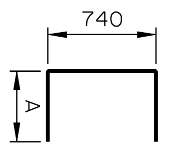


TYPE II

TYPE III

TYPE IV

TYPE BT-72



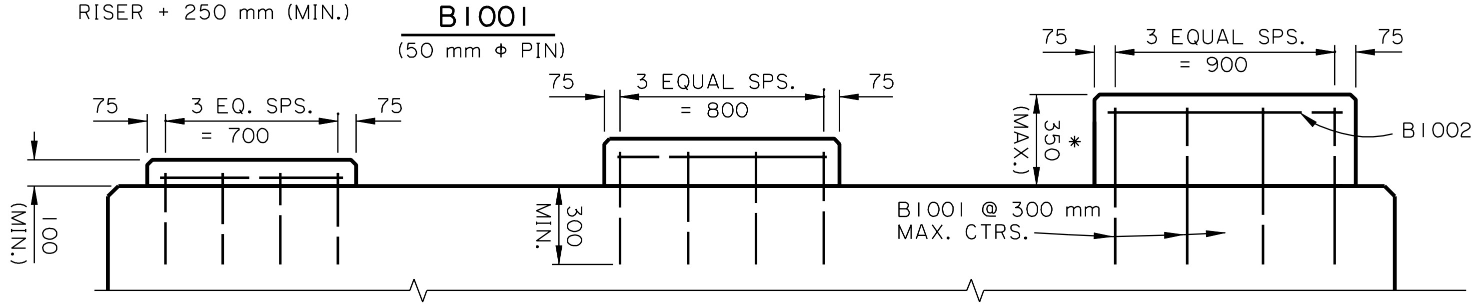
B1001

(50 mm Φ PIN)

DIMENSION "A" =
HEIGHT OF TALLEST
RISER + 250 mm (MIN.)

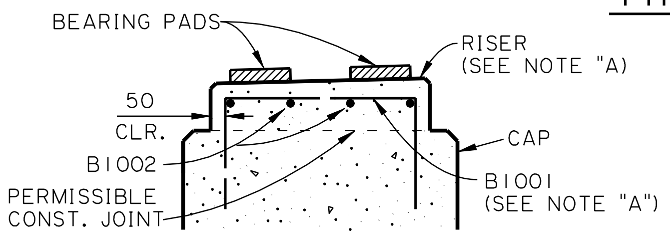
TYPICAL RISER PLANS

(90° CROSSINGS)



TYPICAL RISER ELEVATIONS

* RISER HEIGHT MAY EXCEED 350 mm IN SPECIAL CASES WITH APPROVAL FROM THE BRIDGE ENGINEER.



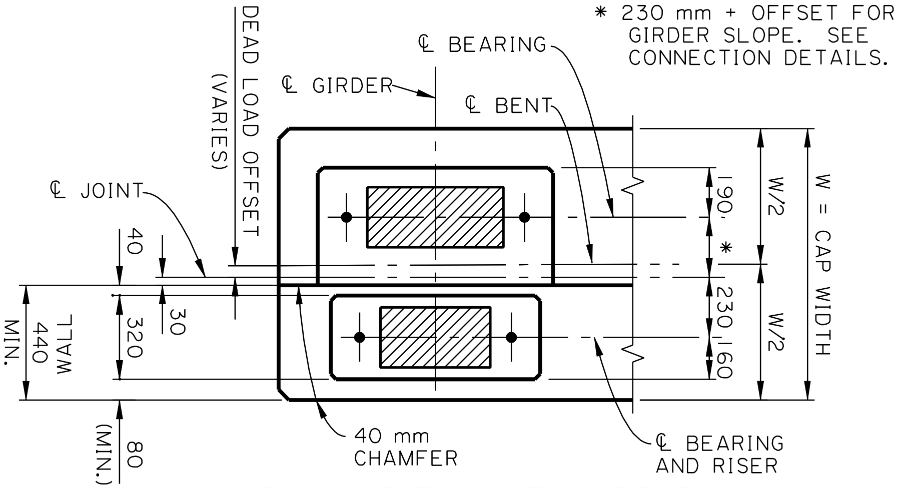
RISER SECTION

NOTE "A"

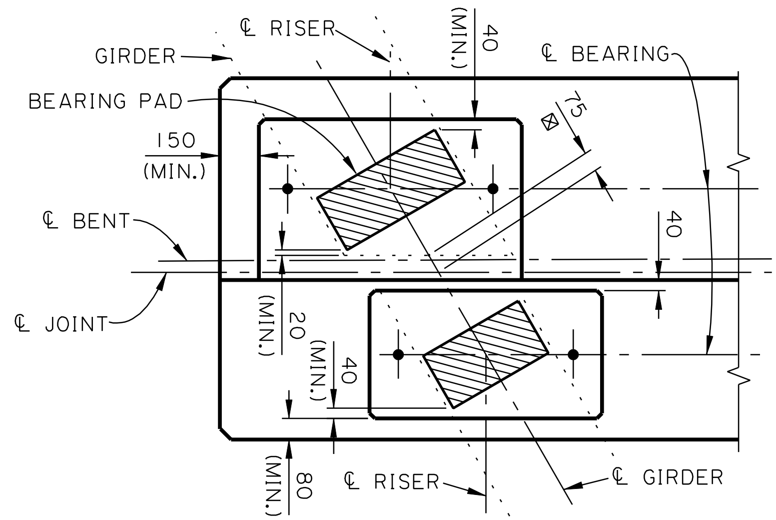
RISERS MUST BE SLOPED WHEN FINISH GRADE EXCEEDS 1%. FIELD ADJUST B1001 TO PARALLEL RISER SLOPE WHEN SLOPE EXCEEDS 3%.

RISERS FOR 90° CROSSING

PILE BENT DETAILS

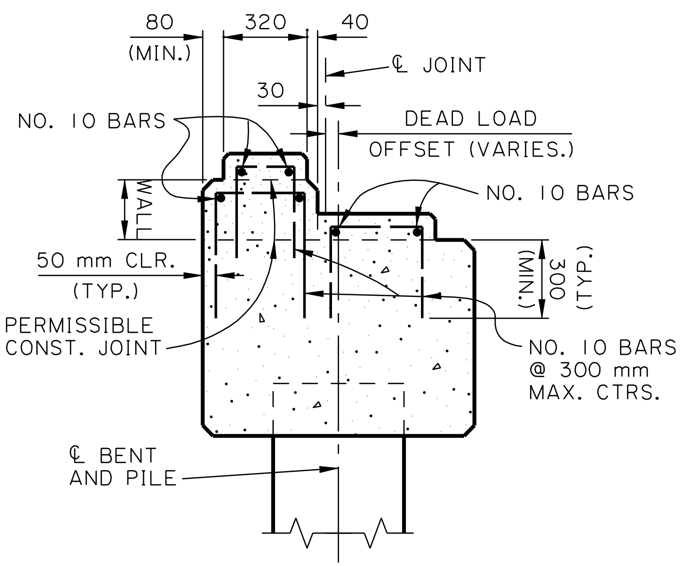


PLAN AT TRANSITION BENT



PLAN AT SKEWED TRANSITION BENT

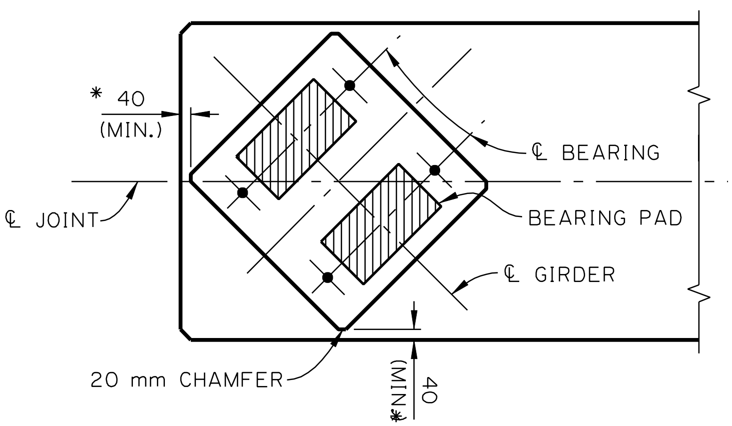
⊠ ADJUST FOR OFFSET DUE TO GIRDER SLOPE. SEE CONNECTION DETAILS.



TYPICAL CAP SECTION AT TRANSITION BENT

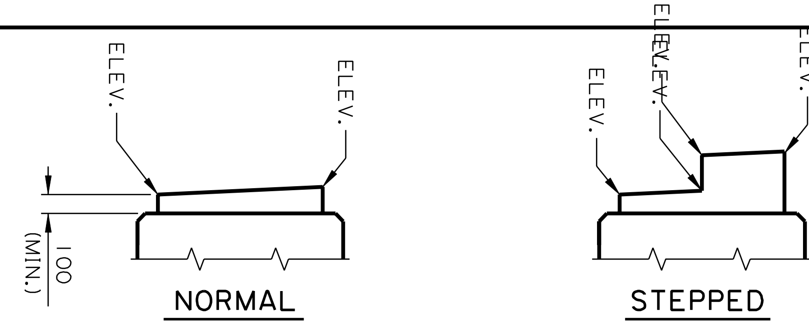
TRANSITION BENTS

PILE BENT DETAILS



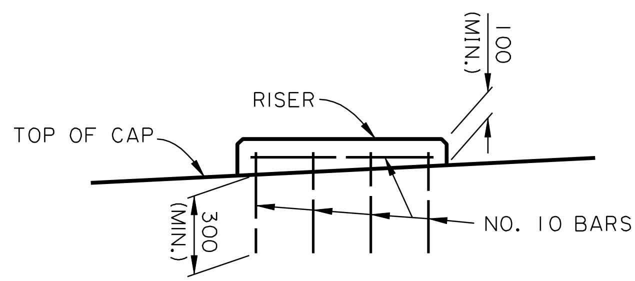
PLAN AT SKEWED BENT

*DO NOT USE THESE CONTROLS TO REDUCE CAP SIZE BELOW SIZE REQUIRED FOR 90° CROSSINGS.

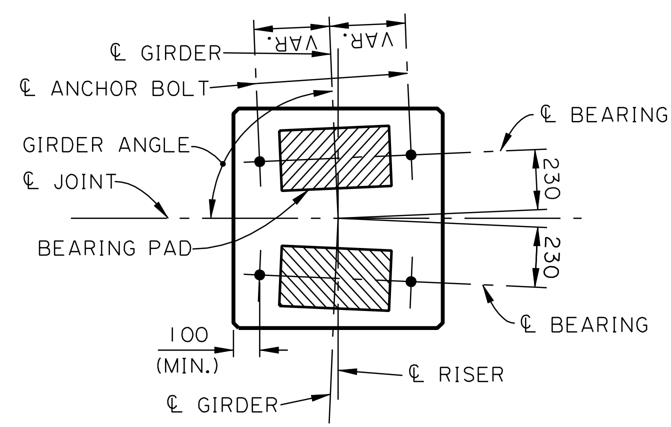


TYPICAL SLOPED RISERS

RISERS MUST BE SLOPED WHEN FINISH GRADE EXCEEDS 1%. SHOW RISER ELEVATIONS AT LOCATIONS NOTED.

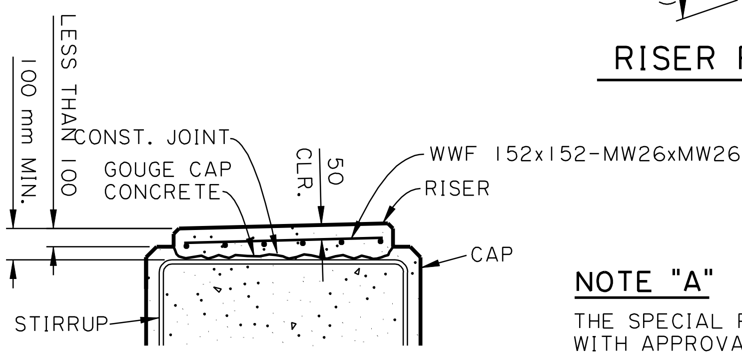


RISER FOR SLOPED CAP



RISER FOR CURVED SPAN

(AND OTHER SLIGHTLY SKEWED GIRDERS)



SPECIAL RISER SECTION

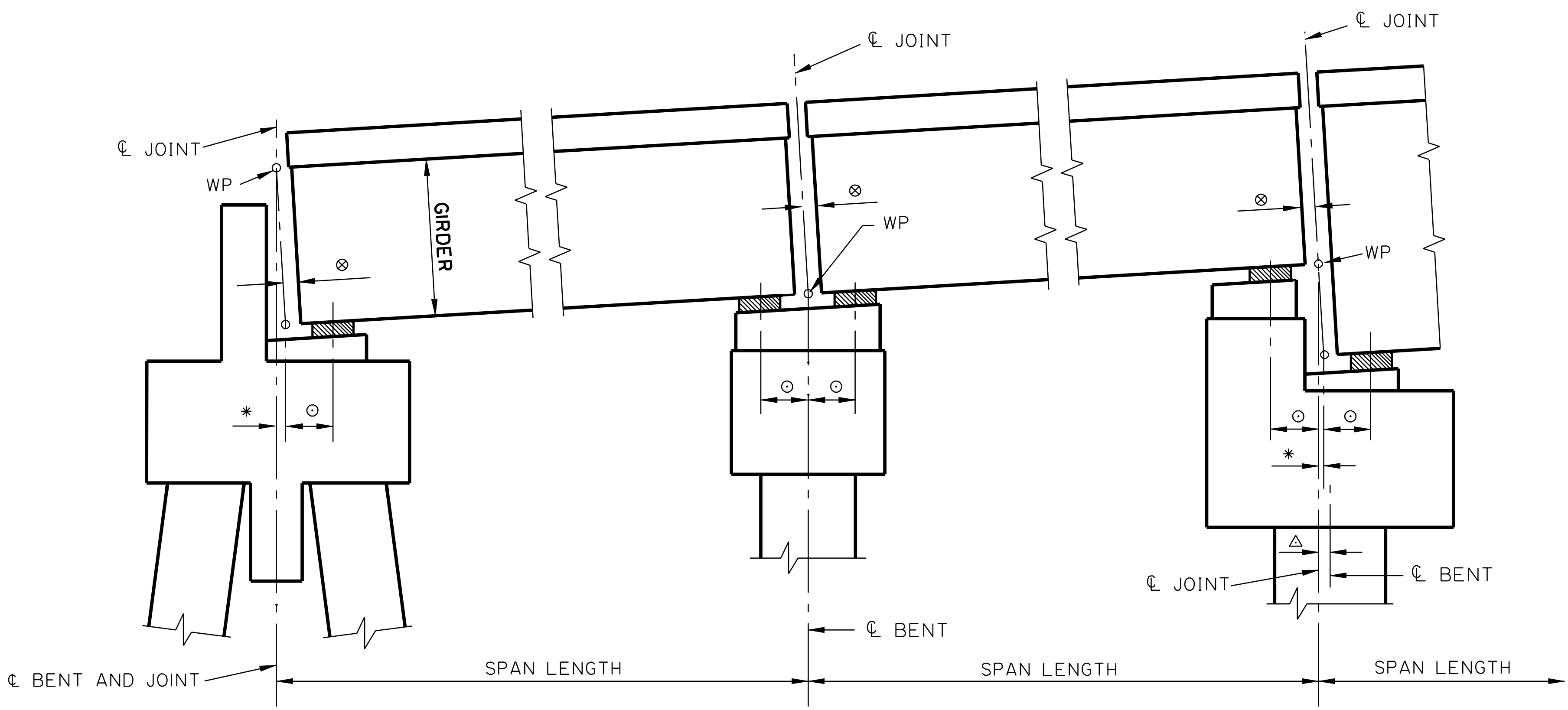
(SEE NOTE "A")

NOTE "A"

THE SPECIAL RISER SECTION MAY BE USED, WITH APPROVAL FROM THE BRIDGE ENGINEER, WHEN AN UNUSUAL SITUATION REQUIRES THE RISER HEIGHT TO BE LESS THAN 100 mm. USE WWF 152x152-MW26xMW26 IN LIEU OF NO. 10 BARS.

MISC. RISER DETAILS

PILE BENT DETAILS



⊙ 230 mm STANDARD BEARING DISTANCE.

∗ SHORTEN GIRDER THIS MUCH.

⊗ 75 mm NORMAL CLEARANCE FOR PRESTRESSED GIRDERS.

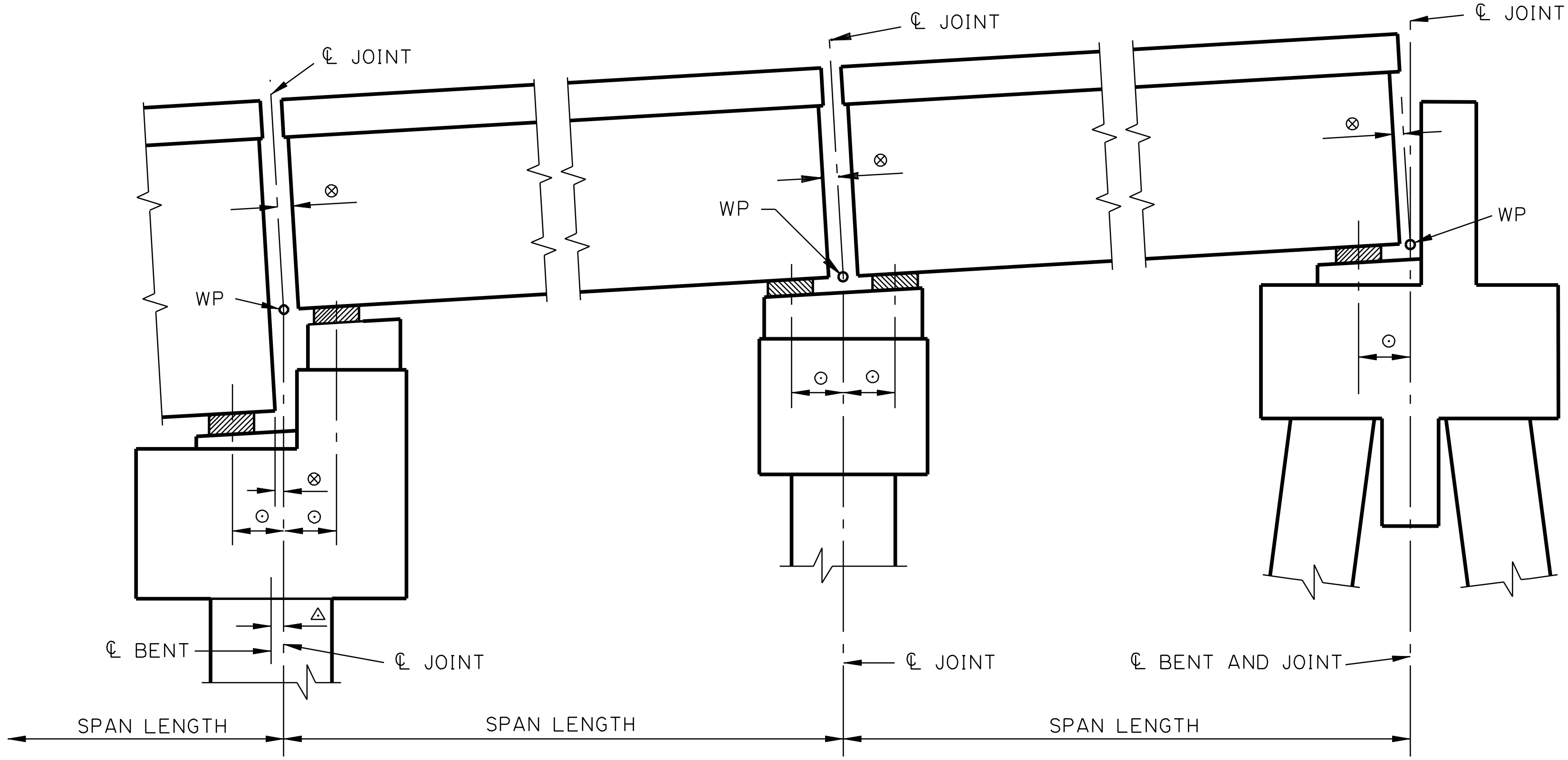
△ DEAD LOAD OFFSET (SEE SHEET 2 OF 2).

WP = WORKING POINT (REFERENCE POINT FOR SPAN LENGTH MEASUREMENT)

BRIDGES ON A VERTICAL GRADE

BRIDGE LAYOUT CONSIDERATIONS

1 OF 2



⊙ 230 mm STANDARD BEARING DISTANCE.

⊗ 75 mm NORMAL, CLEARANCE FOR PRESTRESSED GIRDERS.

△ DEAD LOAD OFFSET (TO COUNTERACT EFFECTS OF UNSYMMETRIC DEAD LOAD).

WP = WORKING POINT (REFERENCE POINT FOR SPAN LENGTH MEASUREMENT).

BRIDGES ON A VERTICAL GRADE

BRIDGE LAYOUT CONSIDERATIONS

2 OF 2

COLUMN BENTS

INTRODUCTION

Column bents are reinforced concrete frames, which are attached to a separate foundation of pile footings, spread footings or drilled shafts. The frame generally consists of columns and a cap and it supports the superstructure. Occasionally, on tall bents, intermediate struts are inserted to reduce the effective length of the column in the transverse direction.

ANALYSIS AND DESIGN DETAILS

Analysis

1. The ratio of unsupported column length to least dimension (diameter) of column ($^L/_d$) should generally be limited to a maximum value of between 9 to 10. Values of greater than 10 will be considered in special situations.

2. Bundled steel will not be used.

3. Unequal dead load reaction from adjacent spans shall be balanced by shifting the centerline of the bent with respect to the centerline of joint, such that there are no appreciable dead load moments caused out of the plane of the bent frame.

4. Live loads from traffic shall be allowed anywhere in the travel roadway and not confined to a design lane.

5. An attempt should be made to place the live load on the span to cause a maximum stress condition in each member to be designed. This is primarily accomplished by trial and error.

6. Columns should be placed under the cap such that the dead load moments induced in the columns are minimized.

7. The number of columns used should be minimized for typical and repetitively used bents. The criteria should be the structural limit of the columns and a proportional size of cap required. Economy and aesthetics will generally be served when the latter is attempted.

8. Circular and square columns will normally be designed as tied columns. Special circumstances may exist where circular columns must be designed as spirally reinforced columns. This method needs the approval of the Bridge Design Engineer. The typical spiral wire reinforcement used with our circular column details does not meet the requirements of a spirally designed column. The spiral reinforcement is only a more desirable method of providing confinement reinforcement, which meets the requirements for the ties in a tied column design. In some cases,

particularly in drilled shafts, contractors may request the use of individual ties in lieu of the spiral cage. This substitution is normally allowed.

9. Careful attention should be paid to stresses caused by temperature and shrinkage when most column lengths get near or below 3.5 m.

10. The foundation for column bents and piers shall be designed so that no piling or drilled shaft goes into tension, except for temporary cases during construction (i.e., cofferdam/steel design) or when designing for extreme events.

11. For bents and piers which require seals, the seal shall be designed for a bond stress of 70 kPa of pile, providing the pile and or soil interaction have been checked for uplift. This shall apply to timber, steel, or concrete piles.

12. In all cases, maximum pile loads shall be determined using the service load design procedure, without live load impact.

Design Details

1. The minimum column diameter shall be 750 mm and larger columns will be in even 150 mm increments.

2. The minimum width of cap will be 100 mm greater than the column diameter or as dictated by the clearance to the bearing areas. The cap dimensions should be incremented in 100 mm increments.

3. The maximum stirrup spacing in a bent cap will be 300 mm. The first stirrup adjacent to the surface of a column will be a maximum of 75 mm away and the first space not greater than 150 mm. The minimum size of stirrups shall be a No. 15 bar.

4. Although designed as a tied column, circular columns shall have 9.5 mm spiral steel at 150 mm pitch with 2½ closed turns top and bottom. This does not meet the design requirements for a spiral design.

5. Columns shall be of solid cross-section.

6. Shrinkage and temperature moments in short columns, resulting from long concrete cap pours, can be relieved by using effective hinge details or staging the cap pours. A more detailed analysis of the structure / soil interaction may also be investigated to determine the actual effects of these moments on the structure.

7. It is advantageous to batter peripheral piles to minimize group action and stabilize the footing against horizontal movement when needed. However, battering should be practiced only when deemed necessary due to economics.

8. On transition bents, a wall and riser should be used when riser height > 350 mm.

9. For steel and concrete piling, the minimum spacing for foundation piling will be 1200 mm center to center or 3 x diameter, whichever is greater. Timber piling shall be spaced at 900 mm minimum centers. A minimum edge distance of 450 mm shall be provided.

10. When square tapered columns are used , a ratio of 1:50 shall be applied to all tapered faces.

11. Except in cases where uplift is anticipated, steel piles and timber piles shall penetrate the bottom of the footing a minimum of 300 mm and cast-in-place and precast piles shall have a 150 mm minimum penetration. In cases where uplift is anticipated the pile footing connection shall be designed.

12. In most situations, the primary layers of reinforcement shall be placed 75 mm above the tops of the piles. For specific conditions, the design may require placing reinforcing steel between the piles.

13. The top of footings will be a minimum of 300 mm below natural ground elevation and 900 mm below the roadway subgrade for footings under the roadway.

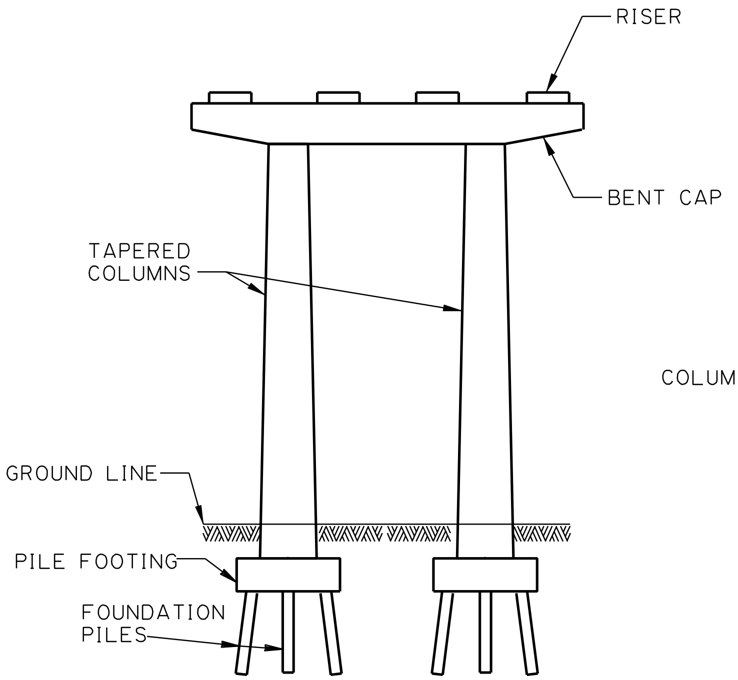
14. Straps should be considered between isolated footings in a column bent when founded on soft soil or where erosion is possible. The primary purpose is to eliminate differential horizontal movement of the footings.

15. Longitudinal skin reinforcement in the vertical faces of caps exceeding 900 mm shall be provided for in accordance with AASHTO Specifications.

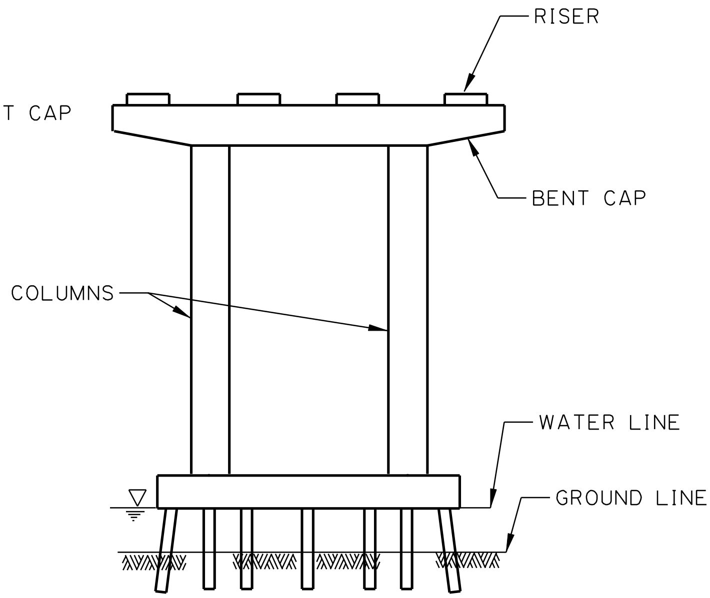
16. For structural mass concrete components whose least dimension exceeds 1200 mm (caps, piers, footings, etc.), minimum reinforcement shall be provided in accordance with AASHTO. This requirement is to reduce the effects of cracking due to the heat of hydration. For extreme situations of massive pours in adverse conditions, other specific counter measures may be required (See ACI). Seal concrete shall not be considered mass concrete.

Typical Column Sizes

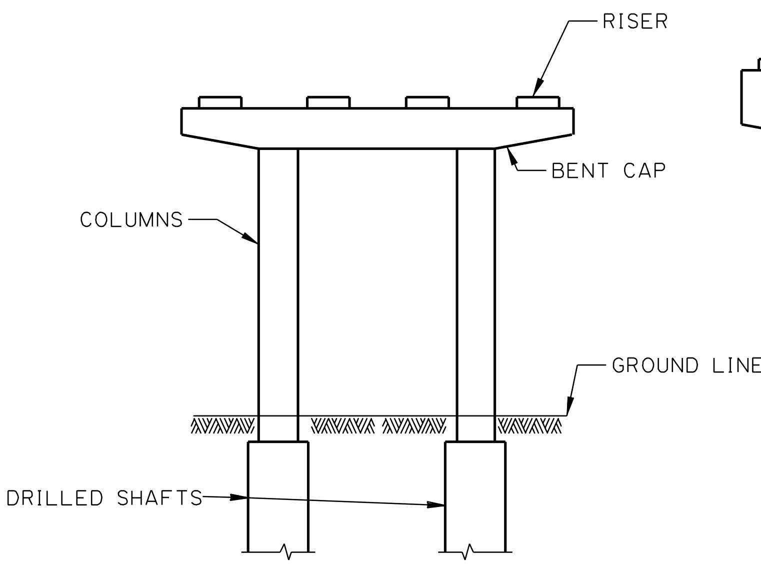
METRIC	ENGLISH EQUIVALENT
750 mm	30"
900 mm	36"
1050 mm	42"
1200 mm	48"
1350 mm	54"
1500 mm	60"



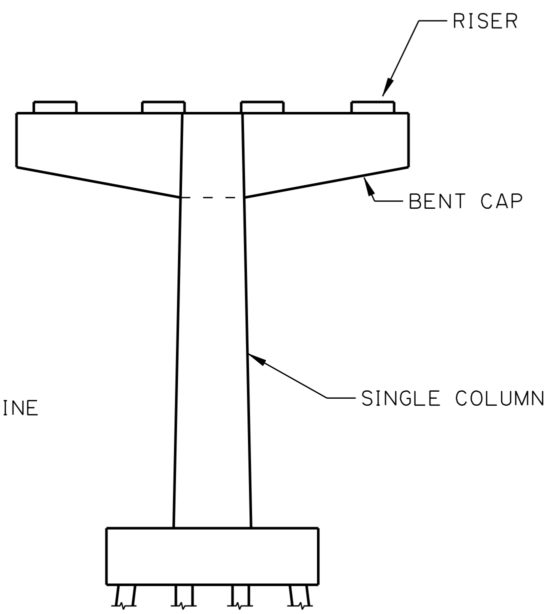
TALL BENT WITH FOOTING



BENT WITH LOW WATER FOOTINGS



BENT WITH DRILLED SHAFTS



HAMMER HEAD

COLUMN BENTS

END BENTS AND APPROACH SLABS

INTRODUCTION

The end bent, as its name suggests, is located at the end of a bridge where the transition is made from the normally deep founded bridge structure to a shallow founded pavement via the approach slab. The typical end bent to be discussed, unless otherwise noted, is constructed of reinforced concrete with a breast wall and wingwalls to contain a soil backfill and pile supported cap. This article is a guide to the analysis criteria and design considerations given to end bents and approach slabs, and is intended as a general policy statement and a supplement to the AASHTO Specifications.

<u>Commentary</u>

The approach slab serves as a transition apron from the soil supported pavement to the pile supported structure. It is intended to smoothly bridge minor differential settlements between the roadway slab and the bridge structure. When large differential long term settlements between the roadway slab and the bridge structure are expected, the Geotechnical Design Section will perform a fill height study to determine if long, flexible, pile supported approach slabs are required. The designer shall furnish the Geotechnical Design Section the required fill height and the time frame of the construction. Two (2) advantages are realized with this solution:

1. A smooth transition is maintained for improved riding characteristics for large expected settlements.

2. The length of conventional bridge structure may be reduced by replacement with the less expensive approach slab.

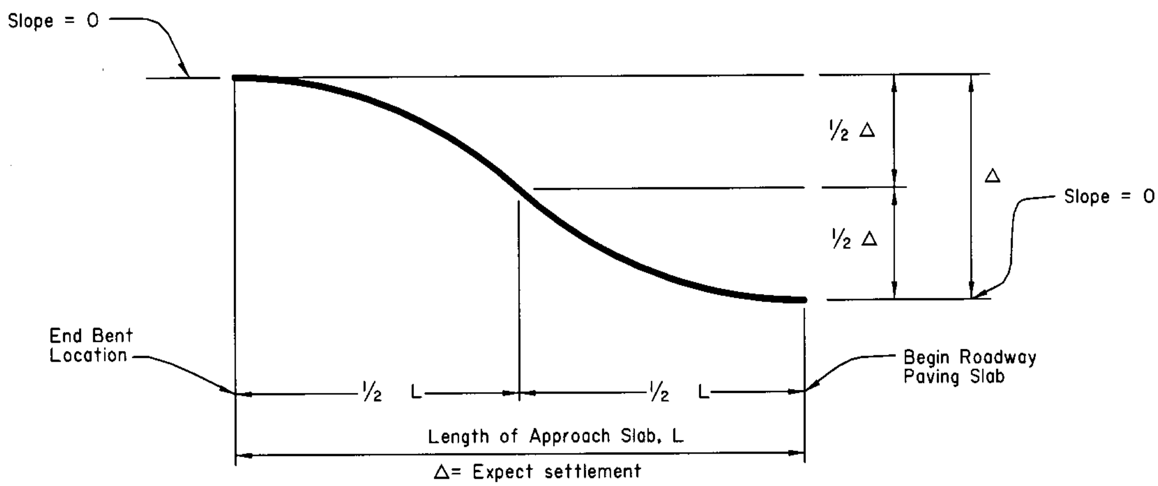
ANALYSIS

1. The short approach slab is founded directly on the paving base and is a one-way slab with its main reinforcement parallel to traffic. No design calculations are required and details are to be in accordance with the guidelines shown.

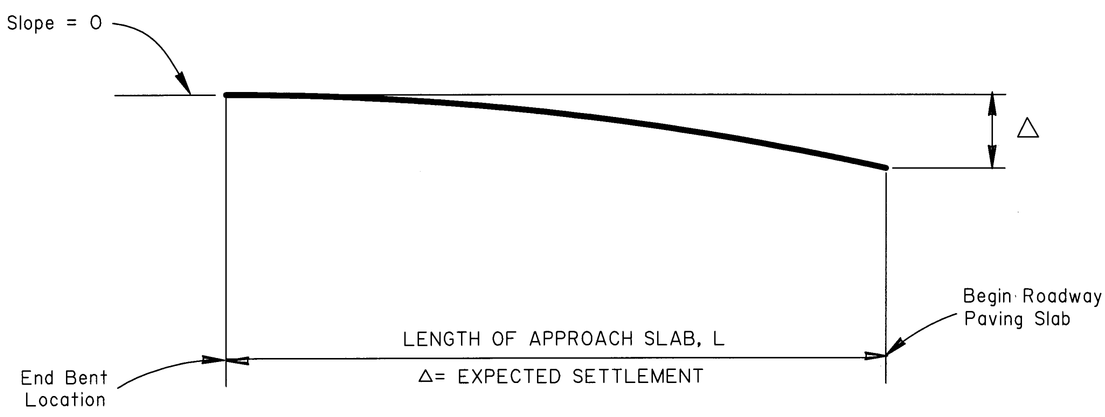
2. The long pile supported approach slab is designed to transition differential settlements between the shallow foundation roadway pavement and the deep foundation bridge structure at the end bent. The total length of the approach slab shall be determined by the Geotechnical Design Section. The permanent deflection due to settlement is assumed to be either a reverse parabola or a single parabola as shown. Piling are usually designed to carry the full live load and dead load near the end bent, and are shortened progressively toward the roadway pavement end of the approach slab. The soil beneath the approach slab should be checked to assure that no piles are tipped in a bearing layer. This configuration is intended to cause a progressive transition from the expected deep foundation behavior to the shallow

foundation behavior. The slab is also designed to carry the additional moment caused by the assumed parabolic deformation.

Condition 1,
Design Deflection for Long Pile Supported Approach Slabs



Condition 2,
Design Deflection for Long Pile Supported Approach Slabs



3. Wingwalls, breast walls and the bent as a unit shall be designed to resist active earth pressure under the appropriate group loads.

4. The reaction of the approach slab to the shelf of the end bent is based on the same assumption as in Items 1 and 2.

DESIGN DETAILS

1. Wingwalls with stabilizing piles are required for all end bents on fills with the exception of slab span bridges and concrete girder spans with Type II girders.

2. Double-row, battered piles are required on all end bents except those for slab spans. Pile batter shall be 1 on 8.

3. On double row pile end bent caps, a 300 mm thick baffle shall extend 600 mm below the cap between rows of piling to prevent the movement of the soil confined by the bent.

4. On skewed or normal end bents, the roadway end of the approach slab is to be squared off for both rigid and flexible pavements.

5. Negative skin friction, caused by the consolidation of the fill and in situ soil in contact with the piles, is assumed to be insignificant because pilot holes are used for piling driven through a compacted fill. Granular material is used to fill the void between the pilot hole and the pile.

6. Pile supported approach slabs are continuous slabs supported by rows of timber piles on 3 m centers. The spacing between the piles in these rows usually varies between two (2) and three (3) meters. The timber piling shall be varied in length from row to row by a constant amount. Piling shall penetrate the footing to resist the tension required to hold the slab in its deformed configuration after settlement has occurred. Pile supported slabs are designed as one way slabs spanning between transverse grade beams at the pile rows.

7. When pile supported approach slabs are used, the barrier rail shall be extended the full length of the approach slab. This will prevent potential guardrail problems caused by embankment settlement in the deep pile region of the approach slab.

8. Prestressed girders end bent caps shall have a depth of 750 mm .

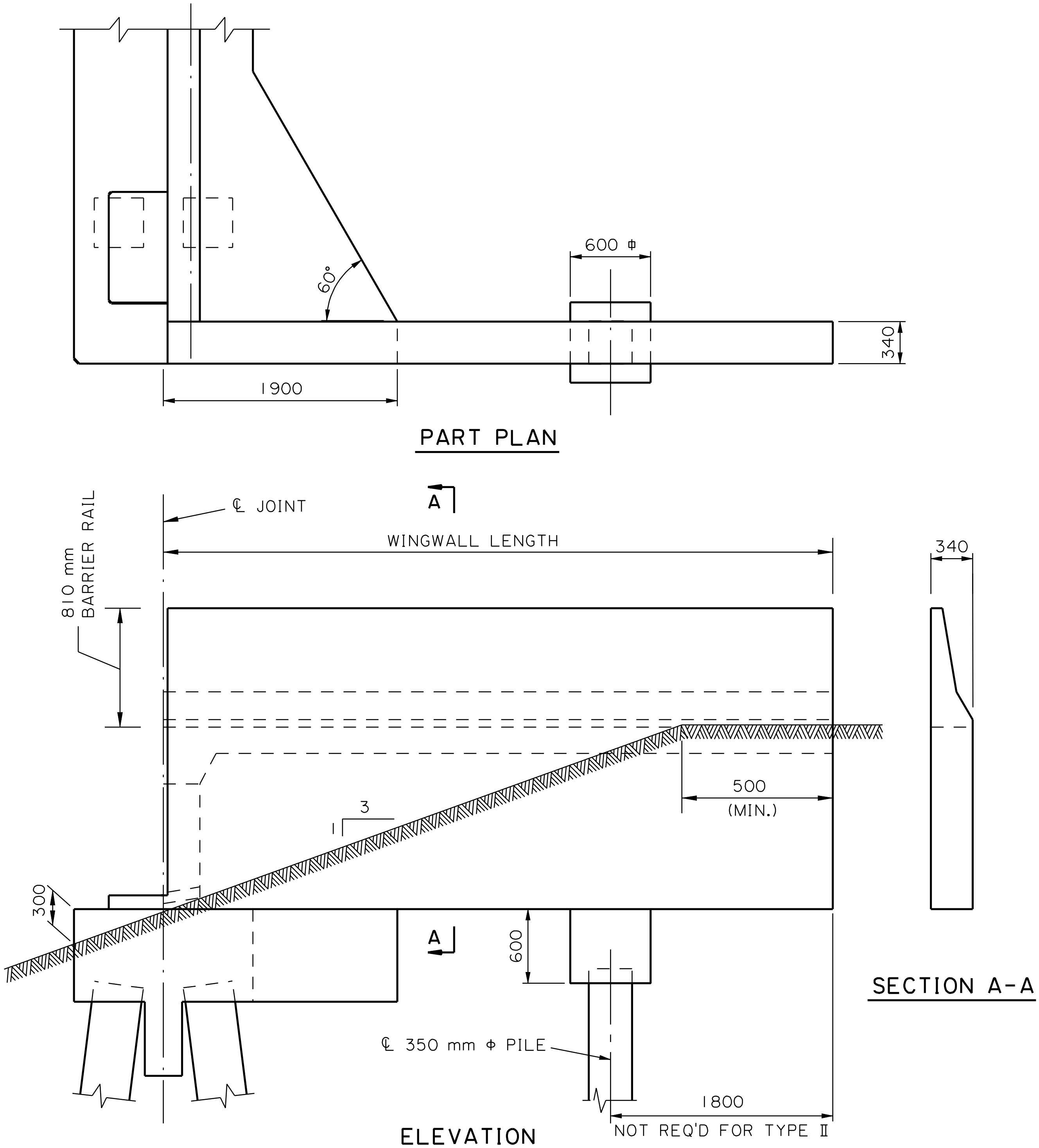
9. In south Louisiana, sand embankments are terminated by a shell plug, which eliminates erosion, formation of cavities, and settlement problems related to sand embankments and sand header banks. In north Louisiana, where shell is not readily

available, sand embankments are to be terminated by a clay plug at structure header banks in the same manner a shell plug is used in south Louisiana.

10. The following criteria will be used in determining slab lengths:

    On fill sections, use 12 m long approach slab.

    On cut sections, use 6 m long approach slab.

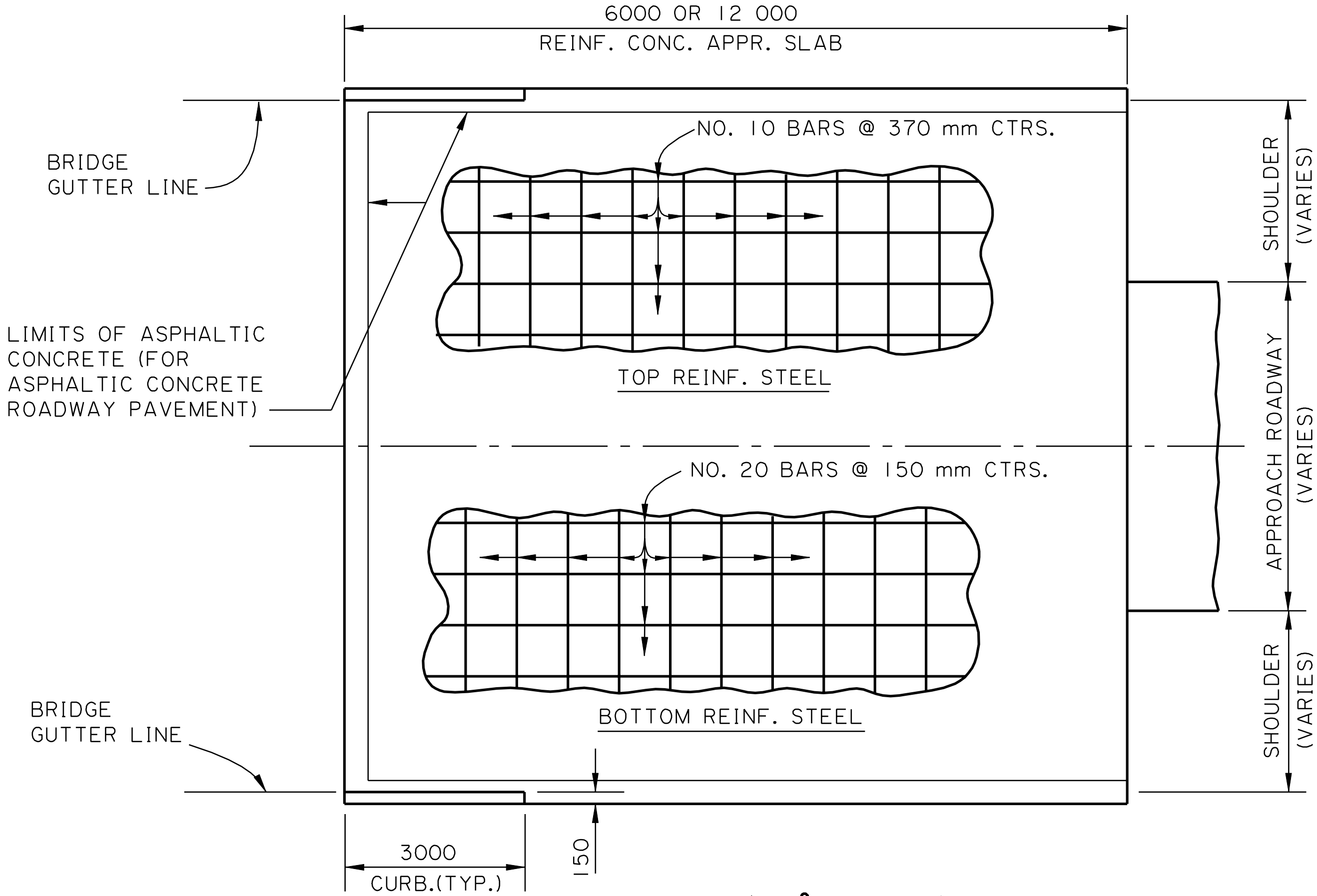


GIRDER TYPE	WINGWALL LENGTH
Ⅱ	4500
Ⅲ	5500
Ⅳ	6000
BT	7500

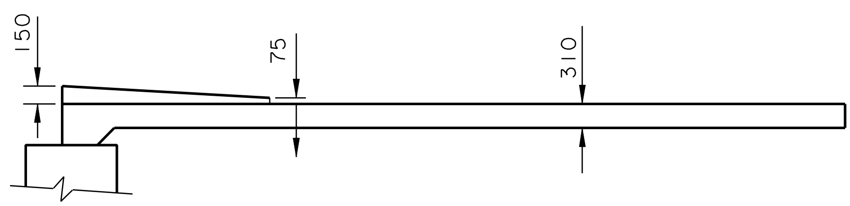
VALUES SHOWN ARE FOR BENTS ON 90° CROSSINGS WITH EXTERIOR RISERS OF 100 mm. ADJUST LENGTHS AS REQUIRED FOR SKEWED BENTS OR THOSE WITH HIGHER RISERS AND ROUND UP TO THE NEAREST 100 mm INCREMENT.

WINGWALL DETAILS

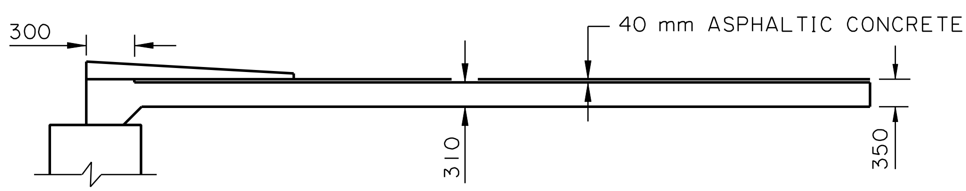
Rev. 06/01/99



PLAN (90° X-ING)



SECTION (FOR PORTLAND CEMENT CONCRETE ROADWAY PAVEMENT)



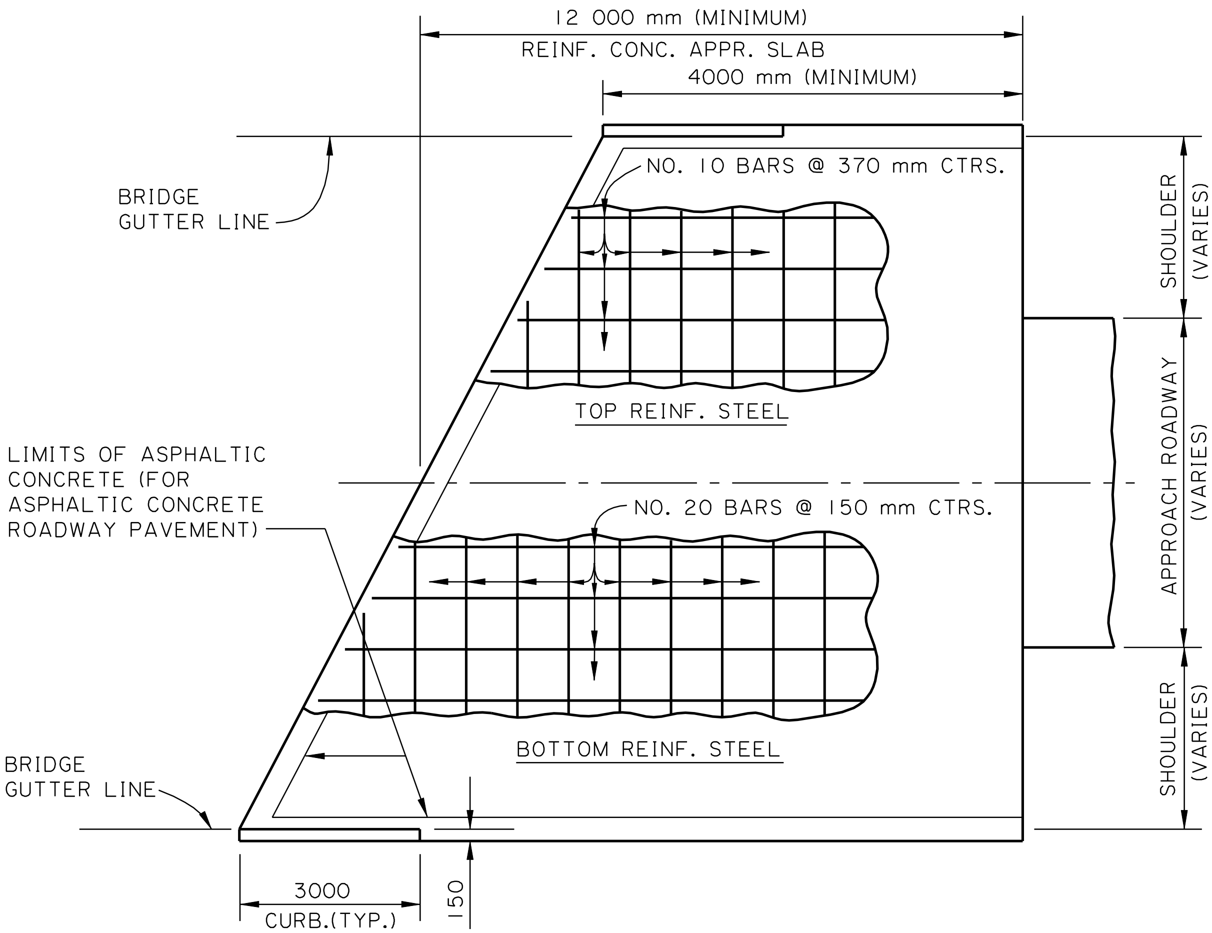
SECTION (FOR ASPHALTIC CONCRETE ROADWAY PAVEMENT)

NOTE:

FOR BEDDING MATERIAL AND DRAINAGE UNDER APPROACH SLAB

SEE STANDARD DETAIL ASD-SS(M).

STANDARD APPROACH SLAB FOR SLAB SPAN BRIDGE



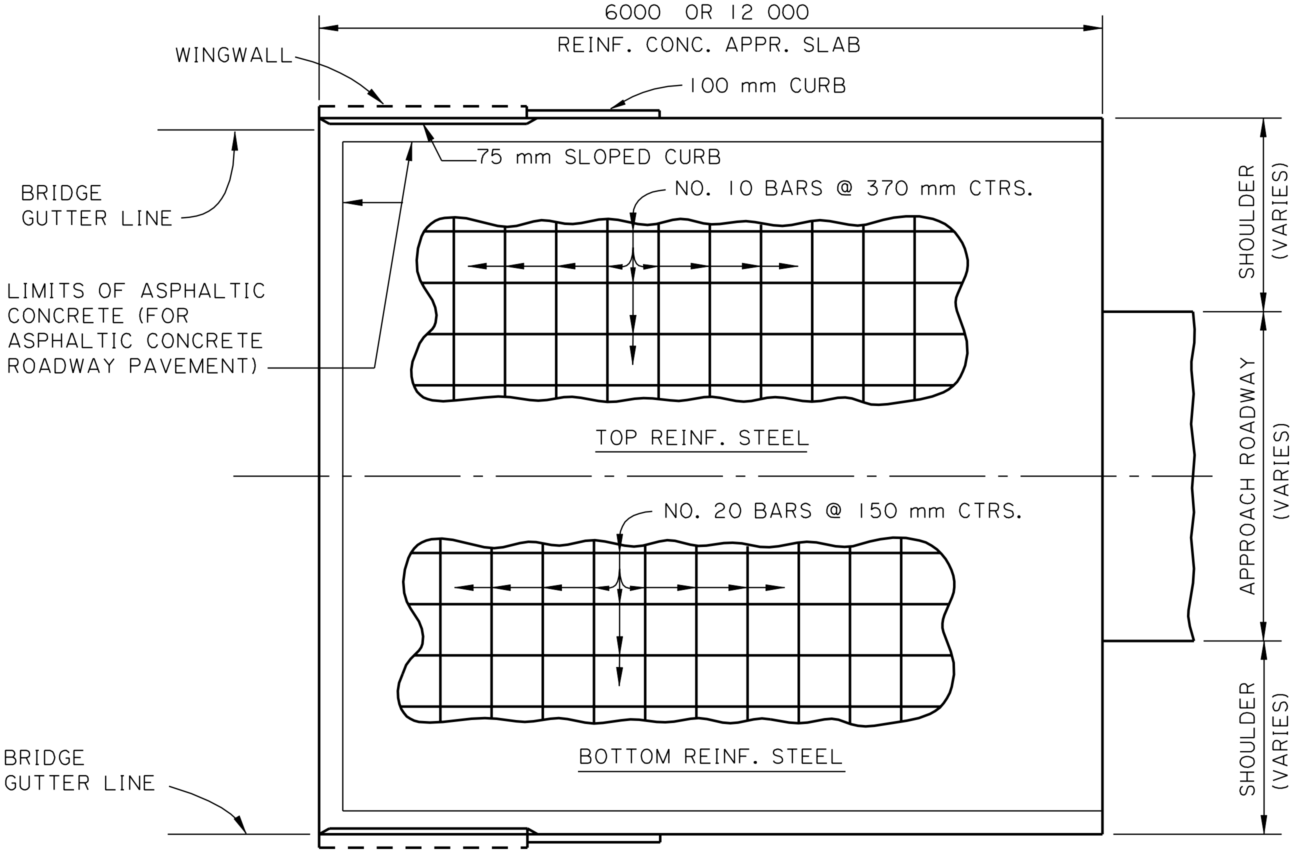
PLAN (SKEWED X-ING)

NOTE:

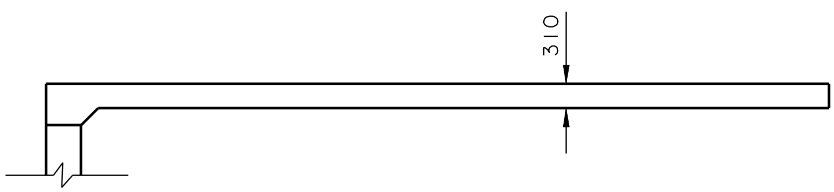
FOR BEDDING MATERIAL AND DRAINAGE
UNDER APPROACH SLAB

SEE STANDARD DETAIL ASD-SS(M).

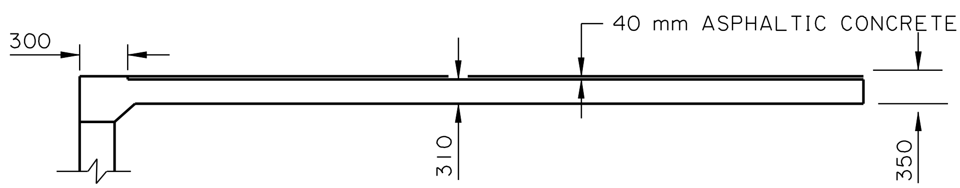
SKEWED APPROACH SLAB
FOR SLAB SPAN BRIDGE



PLAN (90° X-ING)



SECTION (FOR PORTLAND CEMENT CONCRETE ROADWAY PAVEMENT)



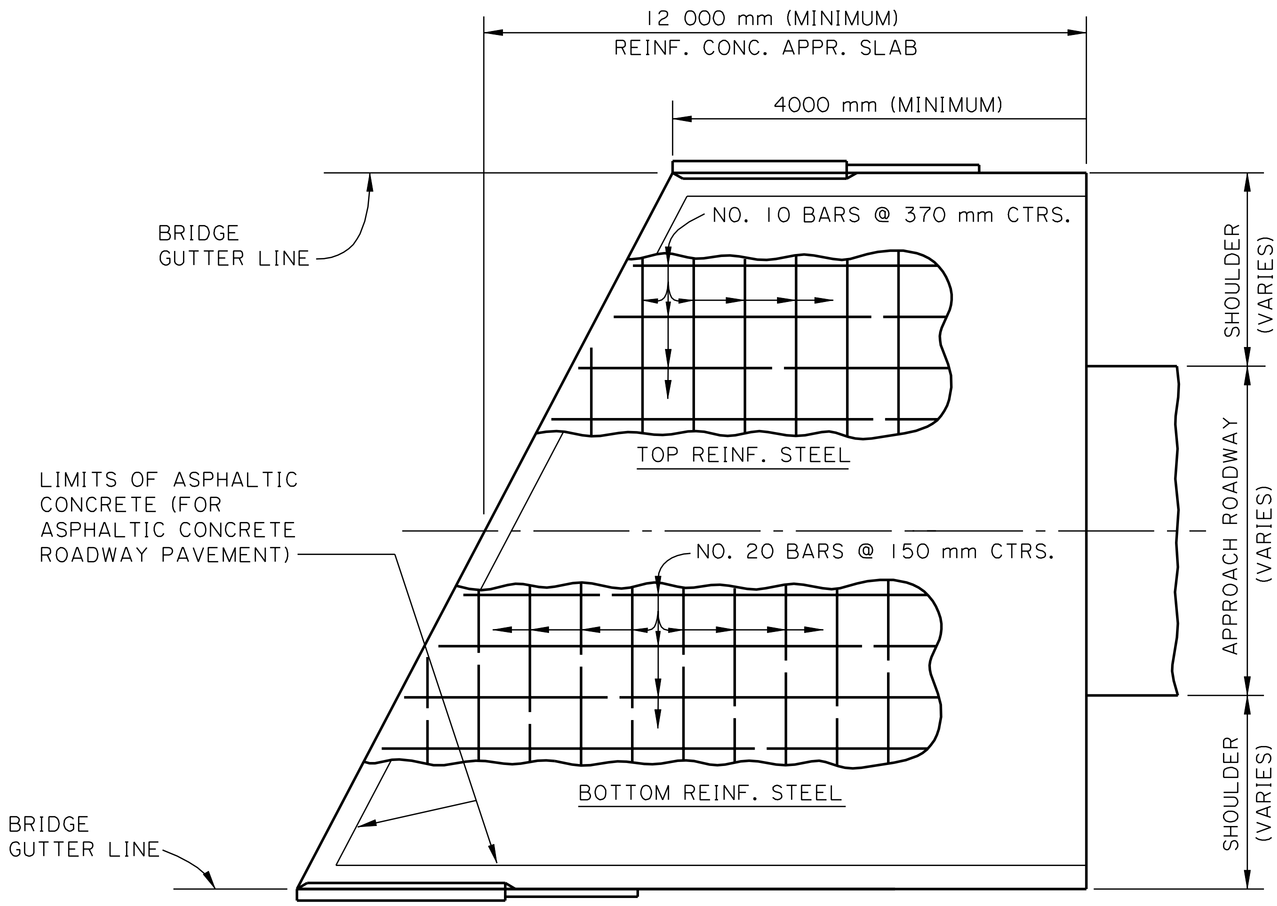
SECTION (FOR ASPHALTIC CONCRETE ROADWAY PAVEMENT)

NOTE:

FOR 75 mm SLOPED CURB, 100 mm CURB AND WINGWALL DETAILS SEE BRIDGE END DRAIN DETAILS.

FOR BEDDING MATERIAL AND DRAINAGE UNDER APPROACH SLAB SEE STANDARD DETAIL ASD-SA(M).

STANDARD APPROACH SLAB
FOR GIRDER SPAN BRIDGE



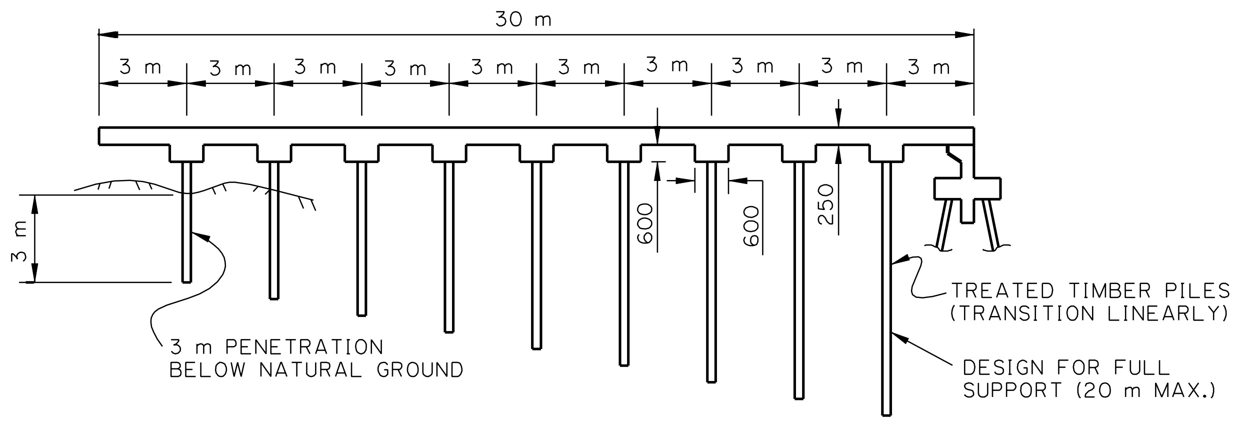
PLAN (SKEWED X-ING)

NOTE:

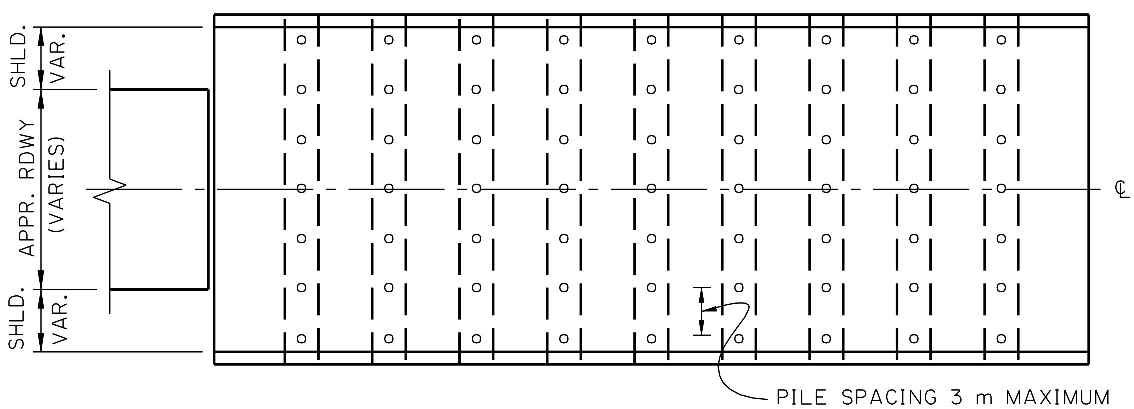
FOR 75 mm SLOPED CURB, 100 mm CURB AND WINGWALL DETAILS SEE BRIDGE END DRAIN DETAILS.

FOR BEDDING MATERIAL AND DRAINAGE UNDER APPROACH SLAB SEE STANDARD DETAIL ASD-SA(M).

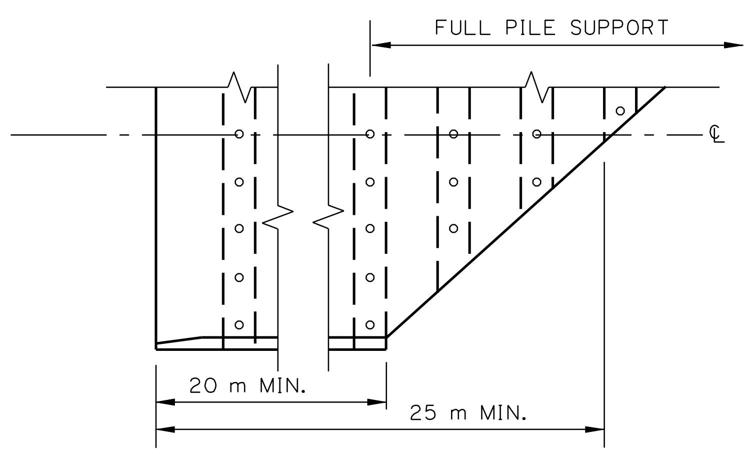
SKEWED APPROACH SLAB FOR GIRDER SPAN BRIDGE



SECTION



PLAN (90° X-ING)



PART PLAN (SKEWED X-ING)

NOTE:

WHEN APPROACH ROADWAY REQUIRES SURCHARGE, INCLUDE APPROACH SLAB AREA TO BRIDGE ENDS.

30 m PILE SUPPORTED APPROACH SLAB

RETAINING WALLS

INTRODUCTION

Retaining walls are used primarily in urban areas to retain embankment slopes which would, otherwise, spill onto adjacent property, or force adjacent ramps or frontage roads outward, thus requiring additional right-of-way. Retaining walls are sometimes connected to the bridge abutment. The breast wall and wingwalls of the end bent is considered a type of a retaining wall since it is retaining the embankment behind the end bent.

Retaining walls are typically either cast-in-place concrete walls or proprietary Mechanically Stabilized Earth (MSE) walls. Typically the MSE wall is found to be most economical in most situations, although certain sites and applications may still require the use of a cast-in-place wall. Cast-in-place walls are placed on either pile footings or spread footings depending on soil conditions and the height of the wall. MSE walls normally require only a small non-reinforced concrete leveling pad for the wall panels or blocks to sit on. The pads are usually 300 mm wide by 150 mm thick.

The bridge section currently maintains standard plan RW-01(M) which is a cast-in-place wall used for minor heights (maximum 1.8 m). The road design section normally uses this standard for minor roadway applications.

CAST-IN-PLACE RETAINING WALLS

Design

1. The working stress method shall be used to design the walls, footings and piles.

2. The Rankine Theory shall be used with the following assumptions:
   Phi = 30 degrees
   Unit soil weight = 1920 Kg/m^3
   Equivalent fluid pressure = 640 Kg/m^3

3. The wall and footing shall be designed for a Class "A" concrete with a f'c = 22 MPa.

4. AASHTO seismic criteria shall be accounted for in the design.

5. Neglect friction between the wall and the backfill.

6. Passive pressure shall be used for resistance only if the fill is high enough on the front of the wall to significantly affect the design.

7. Allowable lateral loads for piles shall be based on information from Chapter 6, Driven Pile Section.

8. Generally, counterforts are recommended for walls in excess of 7.6 m in height. Exceptions should be discussed on a case by case basis. Where counterfort wall sections abut cantilever wall sections, the latter shall have a single counterfort adjacent to the interface.

9. Spread footings are permissible wherever soil conditions, water table elevations and wall heights, are favorable for such footings. A shear key (stub wall) is generally used with spread footings to resist sliding.

10. Timber piles are normally used for retaining walls, however depending on driving conditions, steel or concrete piles or drilled shafts may be required.

11. For spread footings, a factor of safety of 2.0 shall be used for overturning, and 1.5 for sliding.

12. Pile footings shall be designed for no tension in heel piles.

13. The overall stability of slopes in the vicinity of the wall must be checked. This should be coordinated with the Pavement and Geotechnical Design Section

<u>Details</u>

1. Expansion joints should be provided in the wall at approximately 27 m intervals, and contraction joints at 9 m intervals. Expansion joints in the footings shall coincide with joints in the wall.

2. Weepholes for drainage should be provided at 3 m intervals.

3. A geo-composite wall drain should be provided against the backwall with aggregate backfill.

4. A 150 mm diameter perforated pipe drain should generally be placed against the backwall with aggregate backfill.

5. Bridge end drain details with appropriate modifications shall be provided when applicable.

6. In general, stepped footings are permissible. Wall transitions should generally be sloped, except when they support sound walls, in which case they should be stepped

7. If a concrete barrier transition is to be provided, a transition length of 4.6 m should be used. A concrete barrier face is required when the face of the wall is within

9.1 m of the edge of the travel lane. Pedestrian rails or combination rails should be provided for walls more than 0.9 m high if pedestrian traffic is probable.

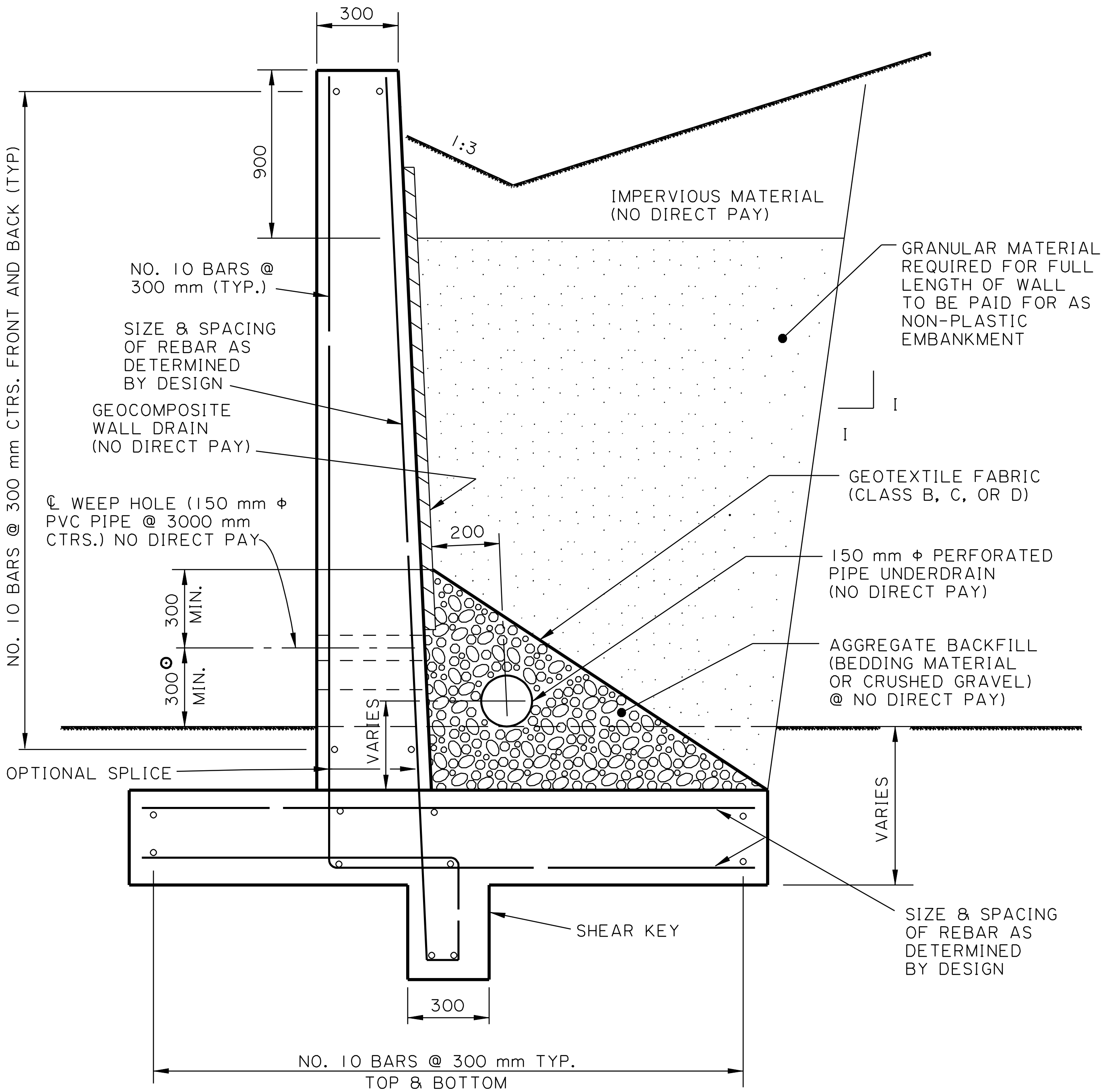
8. Tangent sections of retaining wall, approximately 9 m long, will be allowed in curved sections of roadway with a low degree of curvature.

9. No direct payment will be made for water stop or joint materials.

10. Shear keys should generally be provided at the expansion joints of both walls and footings, and at the contraction joints of wall.

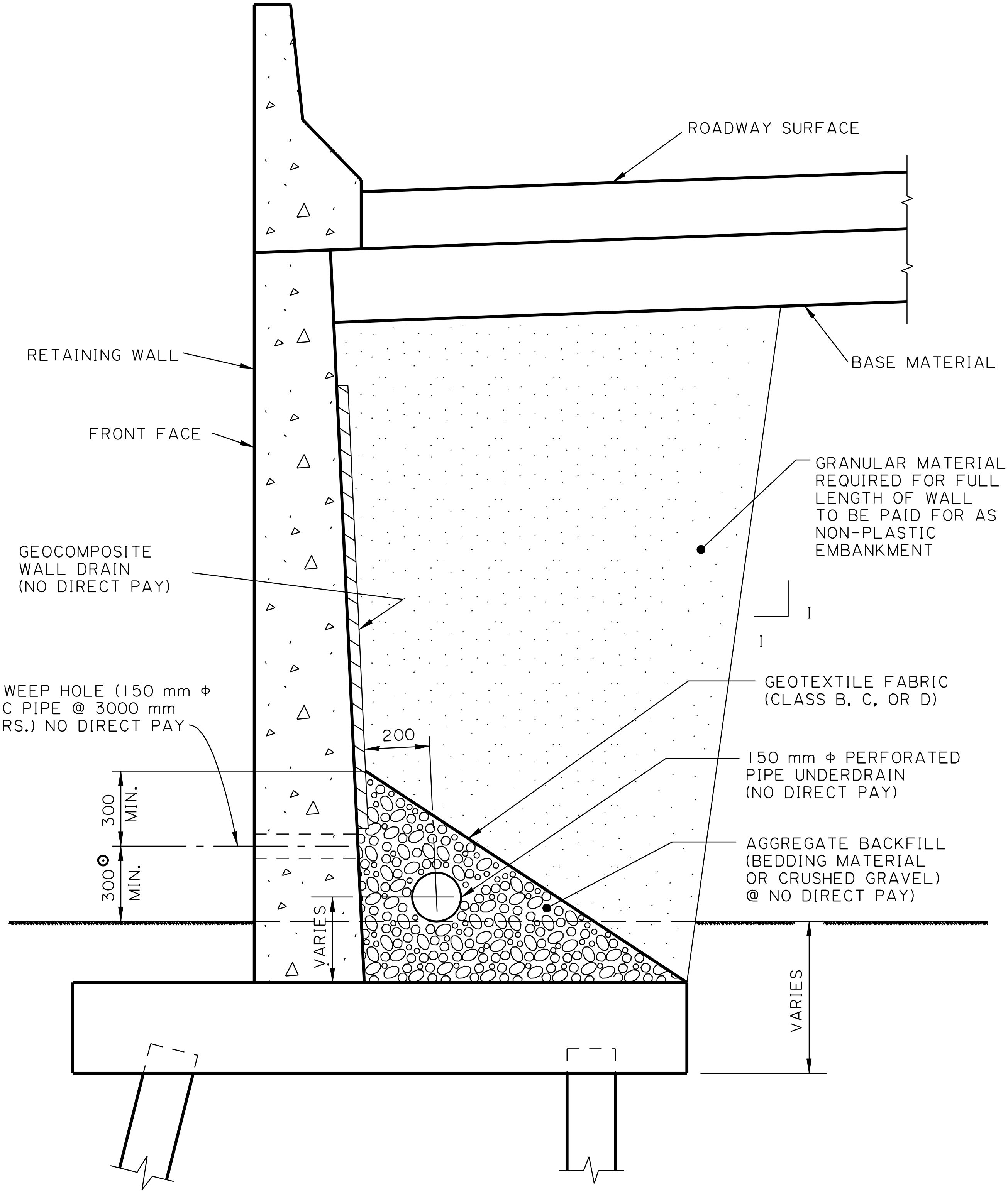
11. Noise barrier requirements shall be accounted for in the detailing when applicable.

12. A pile data sheet that includes pile cutoff, tip, plan pile length or order pile length and maximum pile loads should also be included.



⊙ WEEP HOLES ARE TO BE LOCATED TO DISCHARGE ABOVE GROUND LINE OR GUTTER LINE. EXACT LOCATION TO BE DETERMINED BY THE PROJECT ENGINEER.

TYPICAL DRAINAGE DETAILS AND REINFORCING STEEL FOR CAST-IN-PLACE RETAINING WALL



⊙ WEEP HOLES ARE TO BE LOCATED TO DISCHARGE ABOVE GROUND LINE OR GUTTER LINE. EXACT LOCATION TO BE DETERMINED BY THE PROJECT ENGINEER.

TYPICAL DRAINAGE DETAILS FOR CAST-IN-PLACE RETAINING WALL WITH BARRIER

MECHANICALLY STABILIZED EARTH WALLS

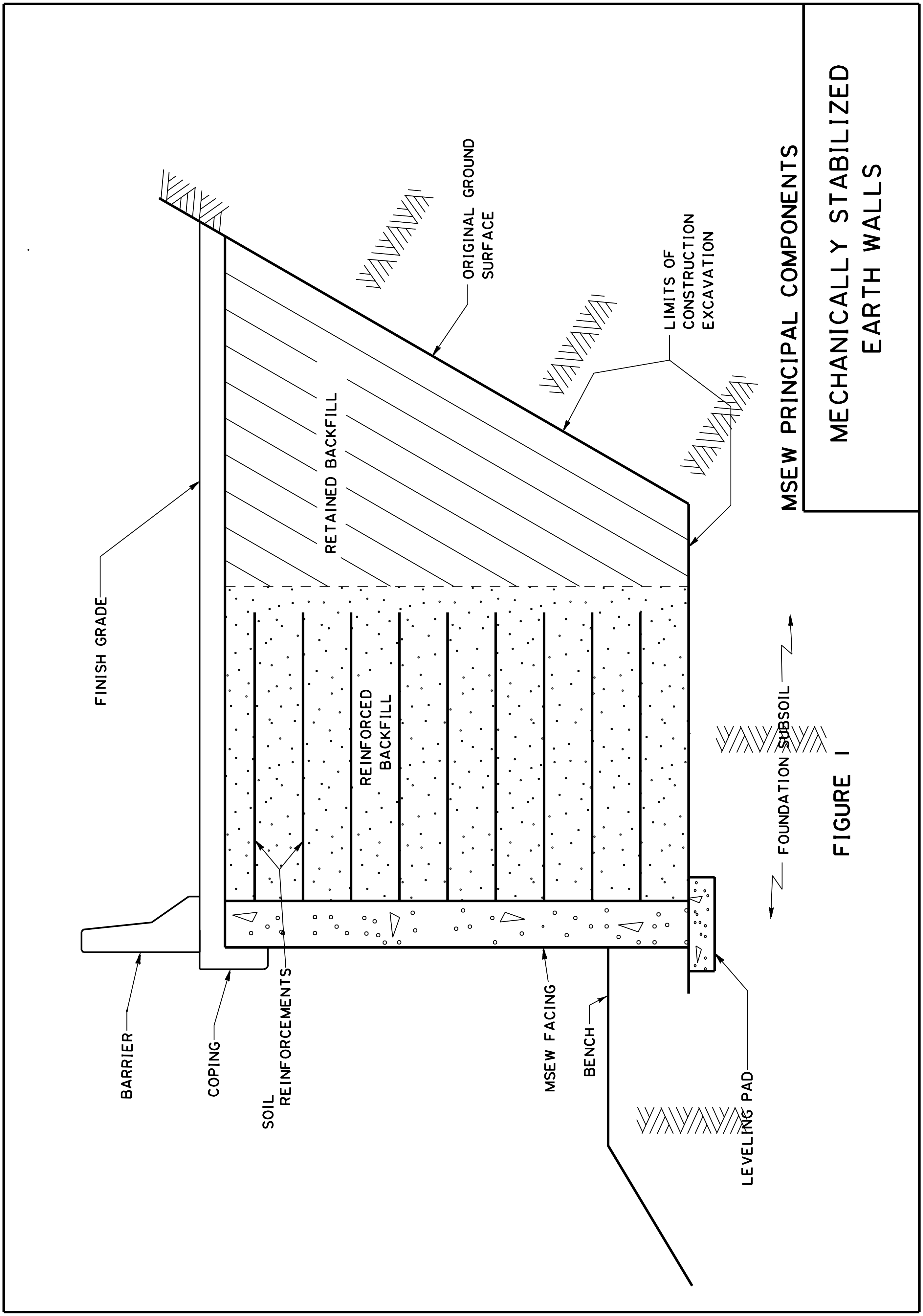
A broad definition of reinforced soil or mechanically stabilized embankments would be the inclusion of reinforcing elements such as straps, bars, welded wire mats, polymer grids, sheets of fabric (geosynthetic) and various anchor systems for the purpose of improving the mechanical properties of the soil mass. All of the system elements including the back fill must receive adequate attention during the design and construction stages.

The concept of soil reinforcement has a well established history dating back to biblical times. However, modern techniques for mechanically stabilizing or reinforcing soil were only introduced about 20 years ago.

Reinforced soil structures are constructed in a manner that produces a structure of alternating layers of soil and reinforcing elements as shown in Figure 1. In general, the spacing between reinforcement layers varies from about 300 mm to 750 mm. Soil reinforcing systems have three main components: reinforcement elements, backfill material and facing elements, see figure 1, page 6 (54). The primary differences between various soil reinforcement systems that are currently available are the materials and configuration of the materials that are used for the reinforcing and facing elements.

Even though different materials are used, the same basic criteria must be employed to design the systems. In general, the basic design procedure for reinforced soil structures is well established. The basic design criteria for reinforced soil retaining walls involves satisfying external stability and internal stability.

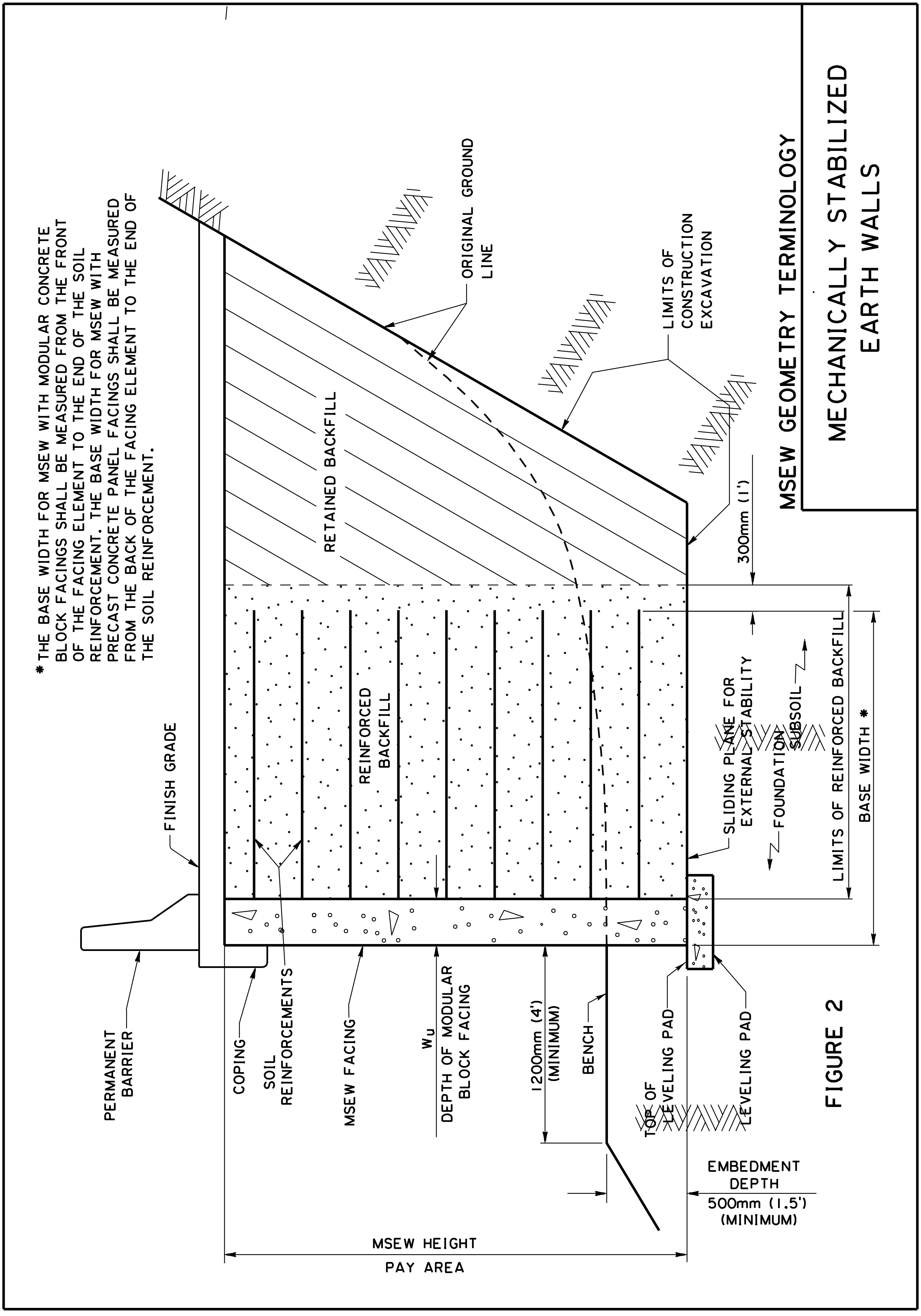
For complete information on this subject, including but not limited to approval procedure for suppliers, design and selection considerations, contracting methods, pay items, details, specific design requirements and construction specifications, please refer to the MSEW Design Guide prepared by the Pavement and Geotechnical Design Section



MSEW PRINCIPAL COMPONENTS

MECHANICALLY STABILIZED EARTH WALLS

FIGURE 1


MSEW GEOMETRY TERMINOLOGY
MECHANICALLY STABILIZED EARTH WALLS
* THE BASE WIDTH FOR MSEW WITH MODULAR CONCRETE BLOCK FACINGS SHALL BE MEASURED FROM THE FRONT OF THE FACING ELEMENT TO THE END OF THE SOIL REINFORCEMENT. THE BASE WIDTH FOR MSEW WITH PRECAST CONCRETE PANEL FACINGS SHALL BE MEASURED FROM THE BACK OF THE FACING ELEMENT TO THE END OF THE SOIL REINFORCEMENT.
ORIGINAL GROUND LINE
LIMITS OF CONSTRUCTION EXCAVATION
RETAINED BACKFILL
300mm (1')
REINFORCED BACKFILL
SLIDING PLANE FOR EXTERNAL STABILITY
FOUNDATION
SUBSOIL
LIMITS OF REINFORCED BACKFILL
BASE WIDTH *
FINISH GRADE
PERMANENT BARRIER
COPING
SOIL REINFORCEMENTS
MSEW FACING
W_u
DEPTH OF MODULAR BLOCK FACING
1200mm (4') (MINIMUM)
BENCH
TOP OF LEVELING PAD
LEVELING PAD
EMBEDMENT DEPTH
500mm (1.5') (MINIMUM)
MSEW HEIGHT
PAY AREA
FIGURE 2

COFFERDAMS

INTRODUCTION

Cofferdams, for the purpose of this manual can be defined as a temporary structure, usually braced steel sheet piling, built to exclude earth and water from an excavation so that work can be performed in a dry condition. Cofferdams should be conservatively designed, soundly constructed and well maintained in order for them to function most often under the most adverse conditions, sometimes including floods and shifting river bottoms. The fact that these structures are temporary, necessitate that they be economically constructed yet the design must take into account all local conditions and make good use of engineering knowledge and judgement.

Most cofferdam systems are composed of two primary components. The sheet pile perimeter with the walers, struts and bracing, and the tremie seal. The tremie seal is a non-structural, non-reinforced layer of concrete poured under water after excavation has been performed to the required elevation. Its purpose is to partially "seal" the bottom of the cofferdam against water seepage and partially or totally counter-act the hydrostatic uplift due to buoyancy. The seal depth is usually a minimum of 1.5 m and can be as thick as 5 m. In some very isolated situations, a tremie seal may not be necessary as in the case of a very hard, non-permeable, clay stratum.

The designer is responsible for the design of the tremie seal and pile/soil interaction. A design water surface elevation must be established after consideration of past historical hydrographs and depth of foundation. The contractor is responsible for the design of the cofferdam sheeting and bracing including length of the sheet piles. The special provisions must include information on the design water elevation and an allowance for the contractor to redesign at a different elevation to accommodate his operations subject to the approval of the Bridge Design Engineer.

DESIGN CRITERIA

1. The total depth of the tremie seal shown in the plans shall be the design depth utilized plus 300 mm.

2. Assume 100% full hydrostatic head acting on the bottom of the seal for the purpose of calculating uplift.

3. Do not use the weight of the sheet piles, struts, etc. in the computation of downward loads. The weight of the structures' piles may be used if properly anchored into the seal.

4. Neglect the friction force between the seal and the sheet piles.

5. 5) Use an allowable 69 KPa friction force between the structural piles and the design seal depth. If more load needs to be developed, anchorage should be designed (AASHTO 1977).

6. Design of the seal and piles in uplift shall be in accordance with Case 1 and 3 of AASHTO 1992 Section 4.5.6.6.2.

7. If pile lengths are being governed by tension, a test pile in tension should be considered to reduce the factor of safety from 3.0 to 2.0.

8. The perimeter of the cofferdam should be sized at least 1 m larger than the footing all around and should also be checked for conflict at the sheet pile/structural battered pile intersection using a realistic sheet pile penetration assumption.

EMBANKMENTS AND REVETMENTS

EMBANKMENT PROTECTION

The following types of embankment protection are generally used for bridges:

1. Cast-in-place revetment is normally used for grade separation structures where hydraulics is not a consideration or above the water line for urban bridges where aesthetics is a consideration. (See Standard Plans on Appendix A)

2. Unless site conditions warrant otherwise, flexible revetment or rip rap shall be specified for all waterways crossings in accordance with the following criteria:

   a) For waterways with design average velocity (v) of 1.0 m/sec. or less, use flexible revetment to a limit of 2 m outside the fascia of the structure.

   b) For velocities between 1.0 m/s and 1.5 m/s, the plans should specify class 25 (kg) rip rap to be wrapped around the embankment on the upstream side to the limits of the approach slab. For certain situations where existing site conditions reveal a history of problems and the design average velocity exceeds 1.5 m/sec., class 55 (kg) or larger rip rap should be specified and wrapped around upstream side as previously noted. Coordination with the District Maintenance and Headquarters Hydraulics sections is recommended as unusual situations arise.

3. Erosion control covering is a system, which should be specified to protect embankments from erosion until vegetation takes over. It may consist of a fiberglass roving or curled wood matting.

4. Spur dikes are occasionally required to break the flow of water around the corner of the bridge embankment in order to prevent abutment scour. Close coordination with the Hydraulics' section is necessary. (See Standard Plans SD-50, SD-100, SD-150, and SD-200)

5. Bridge end drainage is an important aspect of embankment protection. Open deck drains are generally discontinued near the abutments to prevent erosion of the foreslope. In the case of overpasses, or other bridges with vertical curves or relatively high embankments, bridge end drains are normally used. (See Bridge End Drain Standard Detail, Appendix A)

6. Clay blankets or clay or shell plugs are generally required for sand embankments, except when retaining walls are used to contain the sand embankment. In all cases, a select backfill is used adjacent to the abutment See approach slab drainage

standard details, (Appendix A). Embankments and specification thereof are generally covered in the road plans.

The following bridge embankment items should be considered in preliminary planning:

1. When fill heights are a major factor in establishing bridge lengths, a settlement and a stability analysis should be requested from the Geotechnical Design Section at an early stage along with the deep borings. Special construction methods may be employed to minimize the effects of settlement. These may include the use of wick drains, surcharge or prolonging the placement of the approach slab and roadway or a combination of these. In some situations, the Geotechnical Design Section may recommend the installation of settlement plates or other instrumentation to monitor the settlement under roadway embankments or bridge approaches. The results of the settlement monitoring may be used to determine when to remove surcharges or allow paving or construction of approach slabs. If required, the following note shall be placed in the General Notes of the bridge plans: "Settlement Instrumentation: Settlement plates will be required at (locations)".

2. Generally, 1:3 foreslopes will be used for fill heights of 6 m or less. In cases where fill heights exceed 6 m 1:4 foreslopes will be used. Fill heights in excess of 9 m will not be permitted, except with the approval of the Bridge Design Engineer. In cut sections, foreslopes of 1:2½ may be used if soil and hydraulic conditions permit.

3. The designer should exercise caution when placing fill within the channel. This is particularly true when stream velocities are high, or when unstable soil conditions exist, such as “rapid draw down”, in which the water level drops quickly, leaving a heavy, saturated embankment.

4. When placing fill in existing channels, lakes, sloughs, etc., the District Construction Engineer should be contacted to provide probing in order to determine the quality and depth of mucking required. This information will also be useful in investigating the stability of the embankment.

Procedure For Determination Of Toes Of Slope And Computation Of Quantities For Embankment Surfacing Materials

1. When General Plan is drawn the detailer should coordinate with Road Design to obtain roadway typical sections.

2. The General Plan Checker is responsible for checking cross sections sheets for accuracy. The checker is responsible for obtaining the latest roadway typical sections and embankment widening details. Permissible error in toe of slope location is ± one (1) meter except where toe location is critical.

3. Computations for embankment surfacing materials are to be shown on cross sections sheets by the checker using correct toes of slope. These computations are to be back checked by the detailer. A 5% difference between checkers and detailer's quantity will be considered within tolerance. Use the higher number.

SEISMIC REQUIREMENTS

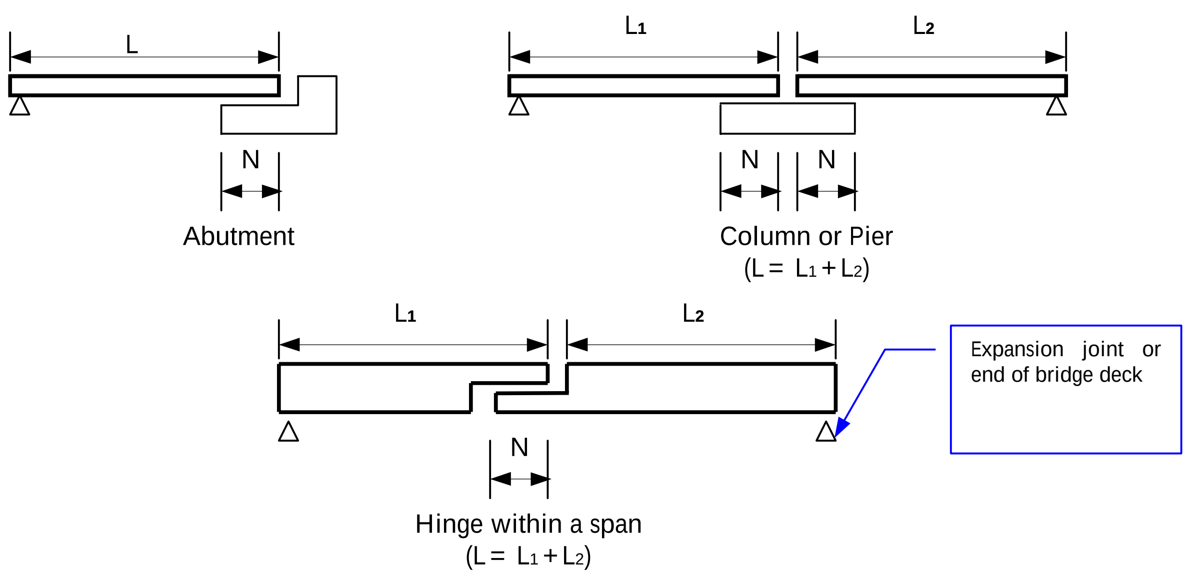
GENERAL

In order to design to resist the effect of earthquake motions, the designer is referred to by AASHTO Standard Specifications for Highway Bridges, Section 3.21 to Division 1-A. The provisions in this section apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 150 m. Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered.

From the contour map of horizontal Acceleration Coefficients (A) provided in AASHTO's Section 3.2, Louisiana has coefficient values that range from about 2 to 4 percent of gravity. Bridges additionally are assigned an Importance Classification (IC) [Section 3.3]. Based on "A" and "IC", all bridges in Louisiana are placed into Seismic Performance Category (SPC) "A" [Section 3.4].

Category "A" requires the least analysis [Section 4.2] and is covered in Section 5. The two requirements which must be met are:

1. Minimum support length
   Provide minimum bearing support length (N) for expansion end of all girders.
   N = Minimum bearing support length for expansion end of all girders in mm.
   = $(203 + 1.67L + 6.66H)(1 + 0.000125S^2)$ where
   L = length (m) as shown
   H = average (m) height of columns supporting the bridge deck to the next expansion joint
   S = angle of skew of support in degrees, measured from a line normal to the span



DIMENSIONS FOR MINIMUM SUPPORT LENGTH REQUIREMENTS

2. Horizontal restraint

A mechanical device used to connect the superstructure to the substructure shall be designed to resist a horizontal seismic force in each restrained direction equal to 0.20 times the dead load reaction at that bearing.

For precast-prestressed concrete girders, the following design values are derived. For other types of superstructures, compliance with the seismic provisions can be accomplished in a similar manner.

a) Allowable Shear Stress:

ITEM	GRADE	SERVICE LOAD (MPa)
Anchor bolts	ASTM F1554 [1]Grade 36	$Fv = {}^{2}1.5 \times ({}^{3}0.40Fy) = 148.8$
Cap Screws	ASTM A307 Grade A	$Fv = {}^{2}1.5 \times ({}^{4}75.8) = 113.8$
Dowels	ASTM A615M Grade 420	$Fv = {}^{2}1.5 \times ({}^{3}0.40Fy) = 252.0$

b) Type of Anchorage:

expansion ends: two 30.0 mm diameter anchor bolts with four M24 cap screws
fixed ends: galvanized #35M deformed reinforcing dowels, 1000 mm long

c) Restraint resistance:

Expansion Ends

Anchor bolts:

$A = 502 \text{ mm}^2$ (root area) [assumed for Service Load]

PR = Horizontal Resistance

$= 148.8 \text{ N/mm}^2 \times (502 \text{ mm}^2) \times 2 \text{ bolts}$

$= 149.4$ kN/anchored girder end

Cap screws:

$A = 452.4 \text{ mm}^2$ (gross area) [T10.32.3A(b)]

$PR = 113.8 \text{ N/mm}^2 \times (452.4 \text{ mm}^2) \times 4 \text{ screws}$

$= 205.9$ kN/anchored girder end

Control ≤ Anchor Bolts ≤ Cap screws

PR = 149.4 kN/anchored girder end

[1] 36Ksi = 248MPa

[2] AASHTO Standard Specifications, Division I-A, Section 5.5

[3] AASHTO Standard Specifications, Division I, Table 10.32.1A

[4] AASHTO Standard Specifications, Division I, Table 10.32.3A

c) Restraint resistance (continued):

Fixed Ends

Dowels:

$A = 1000\ mm^2$ (gross area) [assumed for SL]

$PR = 252\ N/mm^2 \times 1000\ mm^2 \div 2$ ends

PR = 126.0 kN/anchored girder end per dowel

d) Provided restraint:

$\Sigma PR > 0.2 *$ Dead Load of Superstructure at support